NDOR Research Project Number RES-1 (0099) P466
Transportation Research Studies

Design Discharge of Culverts

FINAL REPORT

Kent E. Cordes and Rollin H. Hotchkiss

Department of Civil Engineering
College of Engineering and Technology

W348 Nebraska Hall
Lincoln, Nebraska 68588-0531
Telephone (402) 472-2371
FAX **(402) 472-8934**

Sponsored by the
Nebraska Department of Roads

1500 Nebraska Highway 2
Lincoln, Nebraska 68509-4567
Telephone (402) 479-4337
FAX (402) 479-4325

December 1993

ABSTRACT

The objective of this study was to update the design manual and procedures currently used by the Nebraska Department of Roads (NDOR) Roadway Design Division and to provide consistent design procedures for the Roadway Design and Bridge Divisions to follow. To accomplish these objectives, four tasks were set forth. First, review the current design procedures in the Roadway Design Division and the Bridge Division to gain an in-depth understanding of the procedures each division uses. Next, review the American Association of State Highway Transportation Officials (AASHTO) drainage manuals, which provide guidelines for an agency to follow in developing a design manual. Third, update regional regression equations for the State of Nebraska. Finally, prepare the results of this study, as well as the results of two previous studies, for incorporation into the new design manual.

The biggest concern with the current design procedures used at NDOR is the difference in methods used by the Roadway Design Division (culverts) and the Bridge Division (bridges). The distinction between a bridge and a culvert is purely a structural one: a span of 20 feet or less defines a culvert.,and a span of more than 20 feet defines a bridge. It is conceivable that one division might determine that a bridge was required in a location that the other division found appropriate for a culvert. For this reason, a consistent design procedure is needed for both divisions.

The United States Geological Survey (USGS) regression equations for Nebraska were updated in order to achieve this goal. The original USGS study was completed in 1976, using stream flow data collected through 1972. By using the 19 additional years of data now available to update peak flow predictions obtained by Log Pearson Type III estimation, new, more accurate regression equations were developed. These equations can be used by both divisions for more consistent design procedure and elimination of possibile conflicts.

ACKNOWLEDGMENTS

Primary funding for this research was provided **by** the Nebraska Department of Roads. Financial support for equipment was provided by the University of Nebraska Center for Infrastructure Research. We would like to thank all of the engineers at the Department of Roads for taking the time to help with this research.

We appreciate the assistance of the Natural Resources Commission and the Nebraska Water Resources Department, who provided maps and data on dam sites for the State, and Glen Engel at the **U.S.** Geological Survey, who provided computer files of the gage records, without which this project could not have been completed.

We would like to thank Chuck Wingert and Mike Drain, who worked as undergraduate assistants on this project. Also, Chris Miller was very helpful with solving many computer problems that arose during this study.

K. C. would like to thank Brian McCallum, who encouraged him to pursue his master's degree, culminating with this publication. He also thanks his advisor and coauthor, Dr. Rollin H. Hotchkiss, whose assistance and understanding made this project an enjoyable and valuable experience.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented. The contents **do** not necessarily reflect the official views or policies of the Nebraska Department of Roads. This report does not constitute a standard, specification or regulation.

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LIST OF SYMBOLS

a,b,c, d,e	Parameters of Wakeby distribution
A	Drainage area (mi^2 or acres)
A_c	Contributing drainage area (mi^2)
A_m	Drainage area (mi^2)
C	Coefficient of runoff
CN	Runoff curve number
DA	Drainage area (mi^2)
E	Average annual lake evap. (in)
E, , EL(0.7L)	Control point elevation, 0.7 x length of basin above station
EL(HW)	Elevation of headwater of basin
EL(O)	Elevation at basin outlet
E_r	Rim elevation, highest point
E_s	Elevation of station
FI	Flood index
F_p	Pond & swamp adjustment factor
G	skew coefficient
G_w	weighted skew coefficient
G'	generalized skew coefficient
I	rainfall intensity (in/hr)
$I_{24,2}$	2-year, 24-hour rainfall (in)
$I_{24,50}$	50-year, 24-hour rainfall (in)
K	Frequency factor for return period and skew coefficient
L	Channel length (mi)
L_b	Length of basin (mi)
L_v	Length of valley (mi)
m	rank of discharge events from highest to lowest
MSE_G	Mean-square error of station skew
MSE_G	Mean-square error of generalized skew
n	number of years of record
N	number of events in series
P	Mean annual precipitation (in)
P_i	Precipitation index
Q	discharge (cfs)
Q"	Peak discharge for n-year return period
q_p	Peak discharge
q_u	Unit peak discharge (cfs/mi/in of runoff)
RR	Runoff ratio
S	Sample standard deviation
S	Basin slope (ft/mi)
SN10	10-yr moisture equivalent of snow as of March 15
S_v	Average valley slope (ft/ft)
T	Topographic index
T_1	Mean min. Jan. temperature (°F)
T_2	Mean max. July temperature (°F)
T_3	Normal daily March temperature
t_c	Time of concentration (min)
T_e	Effective topographic index
T_i	Topographic index
T_r	Return period (years)
W	Weighting factor
W_b	Basin width (mi)
X	Annual peak flow or log of annual peak flow
$\overline{X}$	Mean of annual peak flows
μ	Sample mode, Gumbel distrib.
α	Scalar parameter, Gumbel distrib.

Chapter 1

INTRODUCTION

BACKGROUND

Virtually all hydraulic and hydrologic designs require an estimation of peak discharge. Hydraulic structure design is based on a certain return period flow. Return period flow refers to the frequency of a flow of a given magnitude. A 50-year flood, for example, has a two percent chance of occurring in any given year. Culverts and bridges must adequately pass the peak discharge to avoid flooding or failure of the structure. Accurate prediction of return period flows enables the: designers to prescribe the most economical structure consistent with public safety.

The Nebraska Department of Roads (NDOR) determines design jurisdiction using the length of span over the waterway as the criterion. A span of 20 feet or less is considered a culvert, and is assigned to the NDOR Roadway Design Division. A span greater than 20 feet is considered a bridge and is designed by the NDOR Bridge Division. The Bridge Division and the Roadway Design Division use different methods to estimate peak discharges, which may result in discrepancies between their discharge calculations.

OBJECTIVES

The design manual presently used by the NDOR Roadway Design Division does not reflect the most current design procedures. One objective of this study was to update the manual to include discharge estimation methods not formerly available, and to clarify methods already included in the current manual. This project is the third and final in a series of studies to be completed for NDOR. The first study was completed by Riley in 1988, and the second was completed by McCallum in 1992. This study was to incorporate the results of the previous studies, as well as those of the present research, into recommendations for the new design manual.

This study also investigated inconsistencies in design procedures between NDOR Divisions. The objective of this part of the study was to recommend a single, uniform design procedure for both the Roadway Design Division and the Bridge Division. The scope of this study was to update NDOR hydrologic design methods. None of the NDOR *hydraulic* design procedures were updated. Thus, methods used to determine the peak discharge may be changed, but methods for sizing the structure based on the new peak discharge results will remain the same.

METHODS

To review the current design methods used in both the Roadway Design and Bridge Divisions, the author spent a week in each division getting hands-on experience. He worked with several engineers in the Roadway Design Division whose techniques were slightly different, and used actual designs and site data to familiarize himself with NDOR practices. This allowed him to compare his results with those of the NDOR engineers.

The author also reviewed the American Association of State Highway and Transportation Officials drainage manuals (AASHTO, 1991). These manuals give guidelines for development of a drainage design manual. The new edition of the NDOR design manual will be based upon these manuals.

The United States Geological Survey regression equations for the State of Nebraska (Beckman, 1976) were brought up to date, using gage records obtained since publication of the original equations, and new, standardized techniques for regional regression equation development. One of these techniques involved computing a regional iso-line skew map for Nebraska. This map was used to assign weighted skew values to each station, which were then used in the Log Pearson Type III discharge estimation process.

The procedures, results and recommendations of this research are documented in this report. A brief literature review is conducted in Chapter 2, concerned mainly with methods of estimating flood frequencies. Chapter 3 summarizes the results of the two earlier studies by Riley (1988) and McCallum (1992). Chapter 4 reviews the current NDOR design procedures in both the Roadway Design and Bridge Divisions. Chapter 5 describes the procedures used to update the Log Pearson Type III analysis and the regression equations, and presents the results of those efforts. Current and proposed methods are compared in Chapter 6, and conclusions and recommendations of this study are given in Chapter 7.

Chapter 2

LITERATURE REVIEW

This chapter presents a review of literature that is pertinent to the scope of this project. The first section is a review of the American Association of State Highway and Transportation Officials (AASHTO) model drainage design manual. The second section reviews statistical methods used in hydrologic analysis. The third section details the development of regional flow frequency equations. The final section is a review of some previously developed regression equations.

AASHTO DRAINAGE MANUAL

The purpose of the AASHTO drainage manual is to provide a guideline for user agencies to develop their own design manuals. The manual is written in a generic manner, so that the user agency needs only to add its specific policies. The manual provides information on general practices, and gives ideas about what the user agency needs to include as far as policies and procedures. Every aspect of the drainage design process is included in this manual. Since the scope of this project is limited to hydrologic analysis, only chapters pertaining to this will be reviewed in depth.

Hydrology

This section of the AASHTO manual reviews design policies, methods, and descriptions of common procedures, and so is the most important section for the purposes of this project. It makes several suggestions initially which relate to previous chapters. These include suggestions about data collection and documentation. The need for cooperation between the designing agency and other agencies interested or involved in the project is also stressed, to help eliminate costs and save time. The manual describes eight possible methods for estimating peak discharge:

1. Rational Method
2. Watershed regression equations
3. Channel geometry regression equations
4. Log Pearson type III analysis

5. Hydrographs
6. SCS and other unit hydrograph methods
7. Computer programs (HEC-1, TR-20, TR-55, etc.)
8. FEMA flood insurance studies (100-year discharges)

Each of the above methods is described in detail in the manual, along with example problems for each method that show exactly how to determine the parameters and apply them correctly. McCallum (1992) also presents a good discussion of the Rational Method. The watershed regression equations and the Log Pearson Type III method are described in detail later in this chapter.

The selection of design flood recurrence interval should be based on several factors. These factors include traffic flow, potential flood hazard, cost of project, and political considerations. Flood frequencies other than the design flood should also be analyzed to make sure that no unexpected hazards or losses occur.

This chapter also presents a discussion of model calibration for use with computer programs. Calibration involves varying the parameters of a model to match actual stream flow hydrograph measurements. Calibration improves the accuracy of peak flow estimates.

Culverts

AASHTO gives the following definition for a culvert: "A culvert is a structure 20 feet or less in centerline length between the extreme ends of openings for multiple boxes, usually covered with embankment and composed of structural material around the entire perimeter, which is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity [for conveying] surface runoff through the embankment."

The manual makes four policy suggestions regarding culverts:

1. The overtopping flood shall be consistent with the class of highway and the risk involved.
2. Culvert location in both plan and profile shall be investigated to avoid sediment build-up in the barrel.

3. Material selection shall include consideration of service life which includes abrasion and corrosion.
4. Culverts shall be designed to accommodate debris or proper access for debris maintenance.

The manual also lists design criteria, including site characteristics, design limitations, design features, and related designs. Some factors that affect these criteria are topography, climate, soil types, allowable headwater, velocities, storage, and development around the project area.

Flood return periods for design are recommended as follows for various classes of roads:

FEMA mapped floodplain	100-year
Interstate	50-year
Primary highway	25-year
Secondary highway	10-year
Local highway	5-year

Minimum culvert sizes recommended for various classes of roads are listed:

Interstate system	**24** inches
Other systems	18 inches
Side drains or drives	12 inches

The remainder of the **AASHTO** chapter on culverts discusses hydraulic design, and includes discharge equations for different types of control at the culvert. Since this project is concerned with hydrology and not hydraulics, these items will not be reviewed.

Bridges

This chapter gives policy and design guidelines for bridges, which are defined as any structure spanning more than 20 feet. **AASHTO** states that the design flood should be based on risk assessment of local conditions, including traffic patterns, environmental consequences, potential property damage, and flood plain management criteria. The design flood **will** then be used to evaluate hydraulic effects such as backwater elevations, velocities, and scour. The minimum design flood should be based upon roadway overtopping. A "superflood" should also be analyzed to ensure no unforeseen damage is incurred.

The above stated criteria are the only hydrologic aspects of bridge design mentioned. The remainder of the design process is based on hydraulic analysis, and therefore will not be discussed. Additional chapters in the AASHTO manual cover items outside the scope of this study, including energy dissipators, storage facilities, storm drain systems, pump stations, surface water environmental aspects, erosion and sediment control, bank protection, coastal zone situations, construction, maintenance of drainage facilities, and restoration.

METHODS FOR ESTIMATING FLOOD FLOW FREQUENCY

Methods used to evaluate and analyze flood events have changed greatly. When the earliest attempts were made to analyze flood discharges, very little discharge data were available. Consequently, only simple, generalized formulas were possible. As more discharge data became available, the methods grew in both complexity and accuracy. A brief history of the evolution of these methods (Benson, 1962) is presented here.

The earliest methods were empirical formulas, and provided only an estimate of the probable maximum flood. These equations typically take the form:

$$Q=CA^{n} \tag{2.1}$$

where: Q = flood flow

C = a coefficient related to the region

A = drainage area

n = a constant

Such empirical formulas do not take into account the frequency of the event, and so are deficient for use in most design procedures today.

The next step in the evolution of flood analysis equations came when attempts were made to account for flood frequency. Designers realized that the probable maximum **flood** expected was not the most efficient design criterion, so statistical elements were introduced.

The first equations to account for frequency were still empirical formulas such as the Hortan Equation:

$$q=\frac{kT_r^n}{A} \tag{2.2}$$

where: q = discharge (cfs/mi.)

k = constant

T_r = recurrence interval (years)

n = varies with location

A = drainage area (mi.^{2})

This particular equation requires the determination of two empirical coefficients and one hydrologic factor. Because the coefficients remain constant only within small regions, the equation is questionable for large regions.

The next improvement was to include precipitation measures in the equations. One of the most famous in this group, and still widely used, is the Rational Equation. It has the form:

$$Q=CiA \tag{2.3}$$

where: Q = discharge (cfs)

C = runoff coefficient (dimensionless)

i = rainfall intensity (in/hr)

A = drainage area (acres)

This method takes frequency into account in the intensity term and assumes that rainfall frequency equals runoff frequency. The intensity is based on an intensity-duration-frequency curve. This method works well in many different regions. The biggest drawback to the Rational Method is that it is applicable only for small drainage areas.

The most recently developed methods are statistically based and offer the advantage of being derived from actual stream flow records. The stream flow data can be fitted to a probability distribution. Based on this distribution, peak flows for a given exceedence probability can be estimated by relating the measured peak flow to watershed characteristics.

The probability distribution which determines the flood frequency (or exceedence probability) can be determined either by graphically or mathematically fitting the distribution curve to the data. Each method has advantages and disadvantages. The mathematical fit allows for consistency, but the resulting function has no apparent upper limit. The function could be extrapolated well outside of the fitted data without any basis in fact. Conversely, when a graphical fit is performed, the end of the drawn line is generally recognized as the limit of accurate prediction.

Graphical Methods

Graphical methods of fitting a distribution curve to data require the determination of a plotting position for each data point based on recurrence interval and discharge. Depending upon the method that is selected, special types of probability paper have been developed to make these points plot on a straight line. There have been many proposed ways to determine the plotting position. Some of these are listed below (Benson, 1962). In the following equations, T_r is the recurrence interval in years, n is the number of years of record, and m is the rank of the record, with the highest record having a rank of one.

1. The California Method is the simplest. The recurrence interval is given as:

$$T_r = \frac{n}{m} \tag{2.4}$$

This method has several problems. The highest return period that can be estimated is equal to the number of years of record. Therefore, if ten years of record were available at a site, the ten year return period is the maximum that can be calculated. Also, since the probability is the reciprocal of the return period, the lowest event of record has a probability of occurrence of one, which means that it is impossible for an event smaller than this to occur.

2. The Hazen Method attempts to artificially lengthen the record:

$$T_r = \frac{2n}{(2m-1)} \tag{2.5}$$

This gives a return period of approximately 2n for the highest flood of record, and, for example, if ten years of record were analyzed, the largest event would have a probability of occurring in 1 out of 20 years.

3. The plotting position formula used by the USGS was developed in 1946 and is the most widely used method today:

$$T_r = \frac{n+1}{m} \tag{2.6}$$

This is similar to the California method, but it lacks the theoretical problems.

Other graphical methods have been proposed to give plotting positions. However, graphical fitting is not used widely today because of the availability of computer applications that can mathematically fit distributions. These mathematical methods estimate flood peaks for a certain return period independent of the number of points in the data set. Peaks can be determined for several different return periods, and these peaks can be plotted to give the frequency curve using the assumed probability distribution.

Mathematical Methods

Many different distributions have been proposed over the years for flood frequency analysis. Flood frequency data, however, does not conform exactly to any one of these proposed methods. Numerous studies have been done to improve the match between predicted distributions and the hydrologic data.

Other proposed distributions for flood frequency analysis (Riggs, 1968) include the Normal, log-normal, Gumbel, and Log Pearson Type III, and the more recent methods, such as the Wakeby Distribution (Houghton, 1978). These methods are be discussed below.

Normal Distribution

The normal distribution is a common distribution used for many purposes. Fitting a curve to this distribution requires the computation of the sample mean and standard deviation. Using these values and tables of cumulative probabilities (published in most statistics texts), values for discharge can be determined for given exceedence probabilities. This method is not generally used for flood frequency distributions because it is bounded by negative infinity, and negative values are not possible in flood data. Generally, this distribution is of interest in hydrologic studies for other reasons, including assumptions about how errors and residuals are distributed in regression analysis (Neeter, 1990).

Log-normal Distribution

The log-normal distribution is similar to the normal distribution, except that the sample variables have been transformed by taking the logarithm. The data is linearized by this transformation, and negative values are eliminated. This distribution has been found to work well for flood frequency distributions (Bock, 1972).

Gumbel Distribution

Sometimes called the Type I Extreme Value Distribution, this distribution requires the mode and scalar parameters. They are calculated as follows:

$$\frac{1}{\alpha}=\frac{S}{\sigma_N} \tag{2.7}$$

and

$$\mu=\overline{X}-\overline{y_N}/\alpha \tag{2.8}$$

where: μ = mode of sample

α = scalar parameter

X = sample mean

S = sample standard deviation

y_N, σ_N are functions of N (Table 2.1)

N is the sample size

Table 2.1. Means and standard deviations of reduced extremes (Gumbel, 1958).

N	y_N	σ_N
10	0.4952	0.9497
15	0.5128	1.021
20	0.5236	1.063
25	0.5309	1.091
30	0.5362	1.112
35	0.5403	1.128
40	0.5436	1.141
45	0.5463	1.152
50	0.5485	1.161
60	0.5521	1.175
70	0.5548	1.185
80	0.5569	1.194
90	0.5586	1.201
100	0.5600	1.206
200	0.5672	1.236
500	0.5724	1.259
1000	0.5745	1.269

Once the parameters have been computed, the straight line probability is computed by the following equation:

$$X=\mu+y/\alpha \qquad (2.9)$$

The variables are defined above. This distribution has also been evaluated extensively in flood frequency analysis (Bock, 1972; Wallis, 1985).

Wakeby Distribution

The Wakeby distribution is a five-parameter distribution given by the following equation:

$$X=-a(1-F)^{b}+c(1-F)^{-d}+e \qquad (2.10)$$

F is a uniform variate between 0 and 1 that depends on the exceedence probability. The parameters a, b, c, d, and e are determined by regression in the following manner (Houghton, 1978):

1. The equation is rearranged and transformed by taking the logarithms of both sides as below:

$$\log[x_k - e + a(1-F_k)^b] = \log(c) - d * \log(1-F_k) \quad (2.11)$$

2. Initial values are set for a and b. Usually, a=O and b=1. An initial estimate of e is then made, and linear regression is performed over the range of annual flood peaks (x_k) at the gage. A search is performed over the range of e to minimize the sum of squares. This results in estimates of c, d, and e.

3. Using the estimated values for c, d, and e, linear regression is performed again in the reverse direction. This gives estimates for a and b. Using the new values for a and b, step *2* is repeated. Usually, one repetition is sufficient.

Most distributions require estimates of moments, such as mean, standard deviation, and skew. With each higher order moment, more instability and variation is introduced into the equation. This is not a problem with the Wakeby distribution because no moments **are** used to determine the parameters. Therefore, no additional uncertainty is introduced. The use of five parameters instead of two or three, however, makes the process more cumbersome.

It has been pointed out (Houghton, 1978) that the Wakeby distribution can mimic other common distributions, but the inverse may not be true. Wallis (1985) used it and achieved excellent results. A detailed description of experiments and results are given later in this chapter.

Log Pearson Type III Distribution

The Log Pearson Type III (LP3) distribution is widely used. It is the method recommended for determining flood flow frequencies by the Water Resources Council (1981, hereafter referred to as Bulletin 17B). This is **a** three-parameter distribution. The three parameters involved are the mean, standard deviation, and coefficient of skew. These parameters are estimated as follows:

$$\bar{X}=\Sigma\frac{X}{N} \tag{2.12}$$

$$S=[\Sigma\frac{(X-\bar{X})^2}{(N-1)}] \tag{2.13}$$

$$G=\frac{NE\,(X-\bar{X})}{(N-1)(N-2)S^3} \tag{2.14}$$

where:
- X = logarithm of annual peak flows
- N = number of items in data set
- $\bar{X}$ = mean logarithm
- S = standard deviation of logarithms
- G = skew coefficient of logarithms

Since the skew coefficient is highly sensitive to extreme events, a procedure is given in bulletin 17B to weight the skew coefficient with a generalized skew value. The generalized skew is obtained from a generalized skew map published in bulletin 17B, and instructions are also given on how to develop a new skew map. The skew is weighted using the following formula:

$$G_w=\frac{MSE,\,(G)+MSE_G(G')}{MSE_{G'}+MSE\,,} \tag{2.15}$$

where:
- G_w =weighted skew coefficient
- G =station skew
- G' =generalized skew (from map)
- $MSE_{G'}$ =mean-square error of generalized skew
- MSE, =mean-square error of station skew

'he distribution is fitted by the following equation:

$$Log(Q)=\bar{X}+KS \qquad (2.16)$$

where: Q = discharge to estimate

$\bar{X}$ = mean of logarithms of annual peak discharges

L = frequency factor based on skew and return period

S = standard deviation of logarithms

Although this method is the recommended technique for determining flood flow frequencies, it is not without controversy. It has been scrutinized since before bulletin 17B made its recommendations. Some problems with this distribution are discussed in the following section.

Problems with Log Pearson Type III

One of the major concerns of this method involves the use of the skew coefficient. Tests have been performed (Hromadka II, 1993) to determine if the skew coefficient at a site differs significantly from zero. Hromadka used single-station data to test the zero-skew hypothesis at significance levels of 80 and 90 percent and found that it was acceptable at those levels.

Methods used for weighting the skew coefficient have also been investigated (Tasker, 1978). Tasker performed a Monte Carlo simulation to determine the optimum weighting factor for the skew coefficient. The simulation involves generating random numbers from a known distribution, in this case the LP3 distribution. Values for mean and standard deviation were set, and the skew coefficient was varied. Large samples of random numbers were then generated. This type of simulation has an advantage over using actual data in that more records of a given length can be used. Tasker generated 500 samples for each of seven different lengths of record.

Tasker rewrote the Bulletin 17B equation for weighting the skew coefficient as:

$$G'=WG+(1-W)G' \qquad (2.17)$$

where W is the weighting factor. He then used several different methods to determine the value for W. Besides the method recommended by bulletin 17B, he used the computed station skew with no weighting., the generalized skew map skew without the station skew, and a weighting method based on record length. That method is:

$$W = \frac{N}{(N+20)} \tag{2.18}$$

where N is the record length in years

Using each of these procedures to weight the skew, he fit the data generated from the simulation to the LP3 distribution. He obtained the best results using the weighting method that takes into account length of record. He concluded that the weighting procedure recommended by Bulletin 17B often results in worse estimates of population skew than using the station skew itself.

Other studies have disputed the LP3 distribution, suggesting that other distributions actually fit the data better. Bock (1972) performed tests to develop nationwide runoff regression equations for small rural watersheds. For this study, the Gumbel, LP3, and log-nonnal distributions distributions were analyzed. Data was used from 493 gages on watersheds smaller than **25** square miles. Goodness-of-fit tests were performed for each distribution. Compared to values for the 50- and 100-year return periods, LP3 overshot by a factor of two to three. The Gumbel distribution was somewhat better, and the log-normal distribution was very close to the expected results. Bock also used a binomial goodness-of-fit test. This test again showed the LP3 distribution to be the worst fit, and log-normal to be the best. Log-normal was therefore the distribution he used in his study.

A test by Wallis (1985) also shows that LP3 performs poorly against other distributions. He used Monte Carlo experiments to generate random numbers from a LP3 distribution. He fit this data using six different methods, including a variation of the Gumbel known as the Generalized Extreme Value (GEV), LP3 with the skew weighted using several different methods, and the Wakeby distribution described earlier.

Using parameters estimated by each of the six distribution methods, Wallis compared the estimated design floods to the known true values. These experiments showed the Wakeby distribution performing the best, with the smallest confidence limits and the least amount of bias. The LP3 results were the poorest. Because of these results, Wallis recommended a re-evaluation of the procedures given in Bulletin 17B.

A possible source of error in the LP3 method is the underlying assumption that discharge data are random (Creighton, 1993). Creighton examined long-period records for Arizona and found a definite cyclic pattern, leading him to conclude that time-dependent data are not distributed randomly. Therefore, statistical analysis cannot be properly applied to such data.

One of the goals of Bulletin 17B was to provide a uniform technique for determining flood flow frequencies. This goal has been accomplished, even if the distribution is not the best one available. Until other methods or distributions are recommended to replace **LP3,** it will continue to be used, as it is in this report.

REGIONALIZATION IN FLOOD FREQUENCY ANALYSIS

The methods described in the previous section are applicable only where stream flow data is available. To estimate flood flow frequencies at ungaged sites one must use a technique known as regionalization. Regionalization generalizes flood flow frequencies throughout a hydrologically homogeneous region. It effectively extends data points to locations where gages have not been placed. The two methods of regionalization most widely used (Riggs, 1973) are described below.

Index Flood Method

The index flood method (IFM) applies a dimensionless flood flow frequency curve for a region to the estimation of the index flood at a particular site. Dalrymple (1960) outlines the procedures for the IFM as follows:

1. Tabulate peak annual flood data for all gages in the region having more than five years of record.

2. Prepare a bar graph showing the years of record available for each gage in order to readily select an appropriate base period. The longest length of record is typically used as the base period. The remaining records are adjusted to the base period by plotting the peak discharge at the base-period site vs. the peak discharge at the site lacking records, holding the year constant for each coordinate pair. A line is drawn through these points. The slope of the line is the correlation coefficient, which is then multiplied by the discharge of the base-period gage to estimate the discharge at the ungaged site.

3. Use the estimates obtained in step 2 to rank the floods for each gage, with the highest flood being number one.

4. Compute the recurrence interval for each flood. In most cases the graphical method is used. The USGS uses Equation 2.2.

5. Plot frequency curves (discharge vs. frequency) for each station.

6. Test for homogeneity. First divide the 10-year flood by the mean annual flood to obtain the 2.33-year flood. Next, calculate the average 2.33-year flood for each region. Then calculate the adjusted length of record, defined as the number of years that data was collected plus one half the number of years the record was extrapolated. A chart (Figure 2.1) is then used to plot the 10-year flood vs. effective length of record. If the values fall between the station frequency curves determined in step 5, the gaged sites are considered homogeneous and all can be used to develop the regional curve. Gages that fall outside the curves should be included in a different region.

7) Compute the median flood ratios. To perform this step, the flows for various return periods are divided by the mean annual flood. For each recurrence interval, the ratios are averaged. These average values are then plotted against the corresponding probabilities. This is the regional frequency curve.

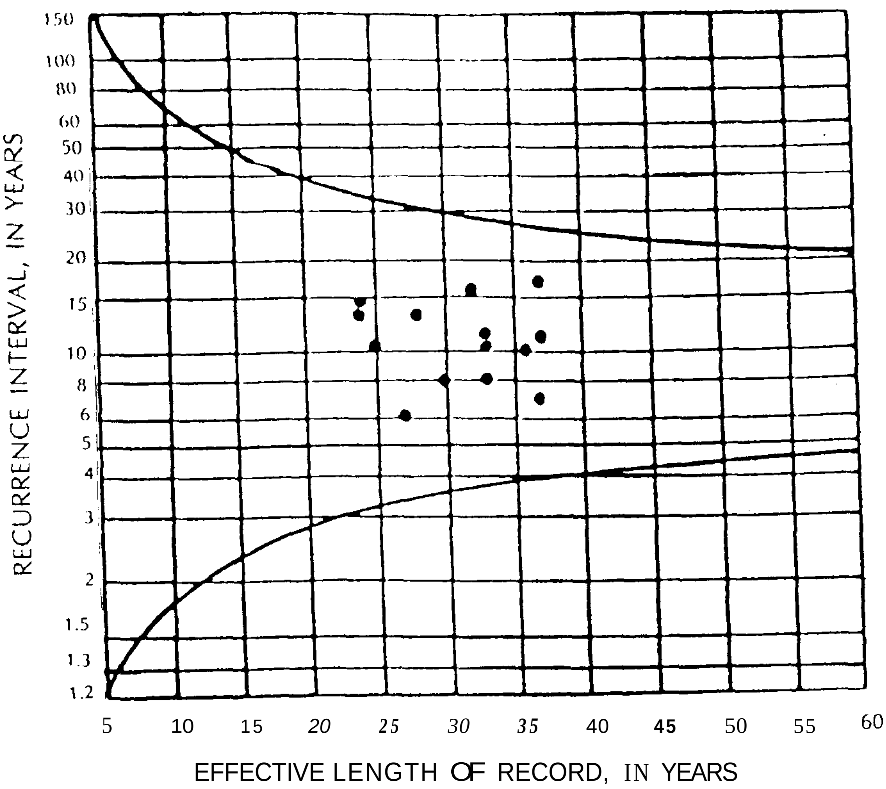


Figure 2.1 Test curves for homogeneity (Dalrymple, 1960)

8) Plot mean annual flows against drainage areas for each gage. The curve defined here allows the estimation of the mean annual flood at a given site.

9) The flow for a particular frequency can be computed by determining the mean annual flow, then comparing the local frequency to the regional flood frequency. The resulting ratio is next multiplied by the known mean annual flood to yield the flow rate for the desired frequency.

The IFM was one of the first attempts to regionalize flood frequency. Many regional equations have been developed using this method. For Nebraska, there are three that cover the state (Reckman, 1962; Patterson, 1966; Matthai, 1968). All three of these methods **are** used by the NDOR Bridge Division to estimate flood frequencies.

Multiple Regression Method

Multiple regression is a technique that relates different flood flow frequencies directly to a stream's physical and climatological characteristics. One equation can be developed for each return period of interest for each region.

To perform a regression analysis, discharges are first estimated for certain return periods at gaging stations. These estimates are then used as dependent variables in the regression analysis. The independent variables are the physical and climatological watershed characteristics. **As** stated earlier, the LP3 method is the recommended technique for determining the dependent variables (flood flows).

Riggs (1973) provides a good background on regression techniques. The regression model typically used in flood frequency analysis is:

$$Q_n = aA^bB^cC^d... \tag{2.19}$$

where Q_n = is the discharge for return period n; a, b, c, d are the parameter estimates of the model; and **A,** B, C are the basin characteristics. The log transformation of this equation is linear. When regression is performed, logarithms are taken of both the dependent and independent variables. The parameters estimated in the regression analysis of the transformed variables can be placed in the form of Equation 2.19. The regression equations can then be applied to ungaged locations by plugging in the basin characteristics for the watershed of interest.

NEBRASKA REGRESSION EQUATIONS

Background

The **USGS** regression equations for Nebraska (Beckman, 1976) were developed using recommendations in WRC Bulletin 15 (1967), which predated Bulletin 17B. These equations, therefore, do not reflect the most up-to-date methods. Both Bulletins 15 and 17B recommend use of the LP3 distribution for estimating flow frequencies at gage locations. However, Bulletin 15 does not cover any aspects of generalized skew coefficient. Beckman's notes indicate that he weighted the skew values for his regression analysis in some way, but his exact procedure is not clear.

Beckman determined the best models for each hydrologic region (Figure 2.2) by using a stepwise regression. He placed a limit of three variables on the model selection to prevent the models from becoming overly complex. He also stipulated that the variables in the models would consist of two physical characteristics and one climatological characteristic. **A** constant value was subtracted from the climatological variables to keep the constant in the equation to a reasonable size. Beckman's equations require that the same independent variables be used for each frequency in a given region to avoid undulations in the computed frequency curve. Equations for 2-, 10-, 50-, and 100-year return periods for each region are shown in Table 2.2.

Table 2.2 USGS regression equations for the five Nebraska regions for 2-, 10-, 50- and 100-year return periods.

Region 1	**Region 2**
$Q_2 = 1.56\ A_c^{0.997}\ (P-13)'^{\ 952}\ L^{-0.794}$	$Q_2 = 0.63\ A_c^{0.797}\ S^{0.427}\ (I_{24,50}-3)^{2.863}$
$Q_{10} = 67.19\ A_c^{0.737}\ (P-13)'^{\ 149}\ L^{-0.608}$	$Q\ -\ 0.49\ A^{0.839}\ S^{0.814}\ (I_{24,50}-3)^{3.320}$
$Q_{50} = 490.86\ A_c^{0.656}\ (P-13)^{0.742}\ L^{-0.543}$	$Q_{50} = 0.51\ A_c^{0.864}\ S^{1.008}\ (I_{24,50}-3)^{3.632}$
$Q_{100} = 996.78\ A_c^{0.624}\ (P-13)^{0.588}\ L^{-0.512}$	$Q_{100} = 0.55\ A_c^{0.872}\ S'^{\ 063}\ (I_{24,50}-3)^{3.731}$
Region 3	**Region 4**
$Q_2 = 103\ A,'^{\ 231}\ (T_3-37)'^{\ 798}\ L^{-1.230}$	$Q_2 = 1774\ A^{1.226}\ (I_{24,50}-5)^{1.831}\ L^{-1.380}$
$Q_{10} = 412\ A,'^{\ 026}\ (T_3-37)^{0.741}\ L^{''948}$	$Q_{10} = 8475\ A^{1.451}\ (I_{24,50}-5)^{1.491}\ L^{-1.783}$
$Q_{50} = 887\ A_c^{0.891}\ (T_3-37)^{0.703}\ L^{-0.745}$	$Q_{50} = 22301\ A^{1.650}\ (I_{24,50}-5)^{1.382}\ L^{-2.081}$
$Q_{100} = 1162\ A_c^{0.843}\ (T_3-37)^{0.686}\ L^{-0.671}$	$Q_{100} = 31454\ A^{1.724}\ (I_{24,50}-5)^{1.365}\ L^{-2.184}$

Region 5
$Q_2 = 0.94\ A_c^{0.831}\ (TI-11)'^{\ 606}\ S^{0.501}$
$Q_{10} = 13.25\ A_c^{0.721}\ (TI-11)'^{\ 114}\ S^{0.443}$
$Q_{50} = 44.07\ A_c^{0.687}\ (TI-11)^{0.845}\ S^{0.521}$
$Q_{100} = 63.87\ A_c^{0.680}\ (T_1-11)^{0.741}\ S^{0.572}$

A_c = contributing drainage area (mi^2); **A** = total drainage area (mi^2); P = average annual precipitation (in.); L = basin length (mi.); **S** = slope (ft/mi) between 0.1 and 0.85 basin length above outlet; $I_{24,50}$ = 50-yr, **24-hr** rainfall (in.); T_3 = normal daily March temperature (°F); T_1 = mean minimum January temperature (°F).

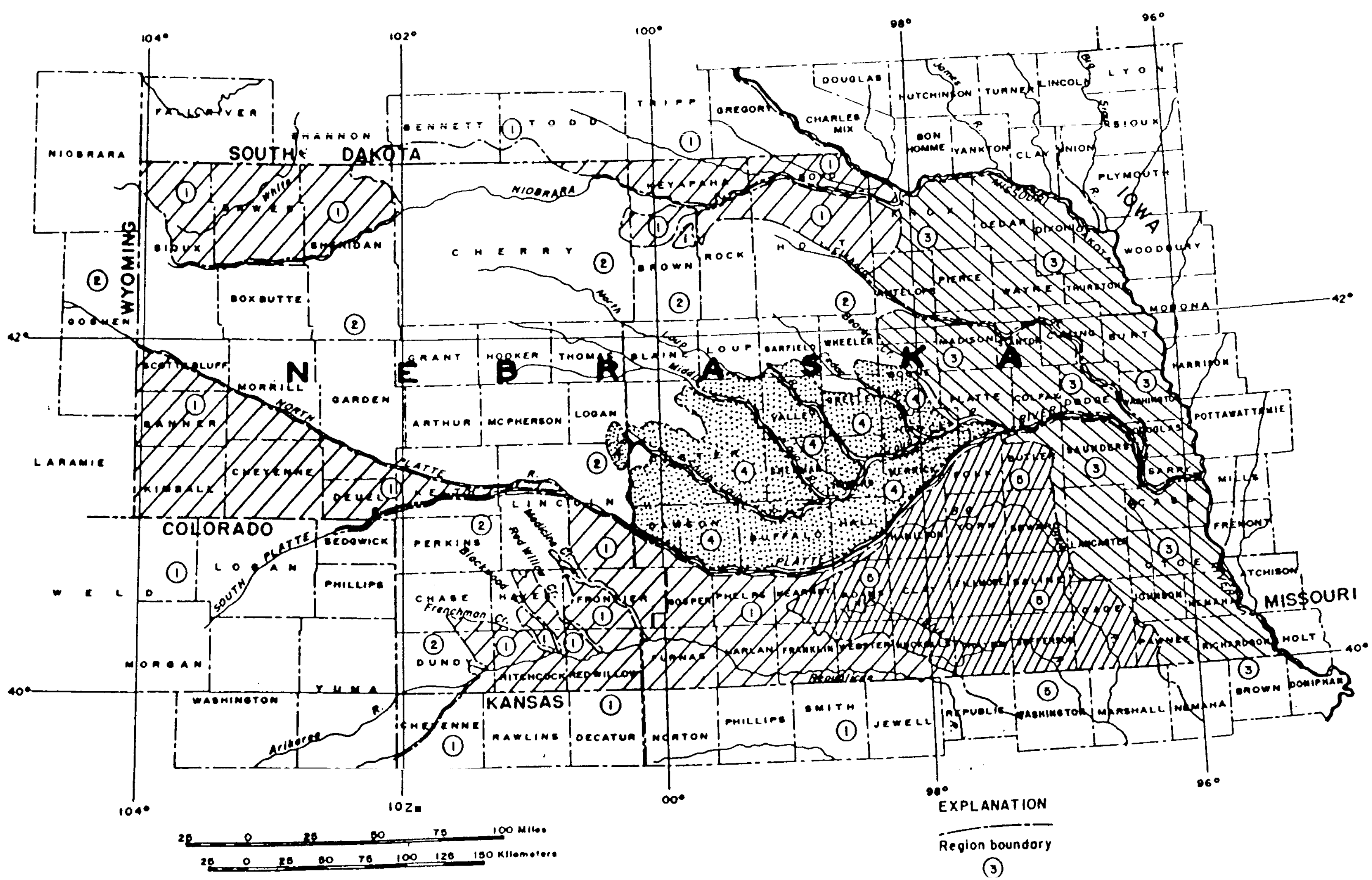


Figure 2.2. USGS hydrologic regions of Nebraska (Beckman, 1976)

Justification for Revising Regression Equations

A major goal of this project was to update procedures within NDOR for hydrologic calculations. The **USGS** regression equations are apparently inferior to more recent methods, but the question remained whether improvements were great enough to justify the expense of developing new equations. Hardison (1971) developed statistical tests to determine the equivalent years of record required to improve the estimates from a gage location. This process could not be applied to Beckman's study because several statistics required for the test were no longer available. These included average skew in each region, average interstation correlation coefficient, and the standard error of estimate for each equation.

However, considerations other than statistical analysis do justify updating the equations. First, the equations do not reflect current procedures, especially in the area of skew weighting. Second, the standard error of the skew coefficient is strictly a function of record length (Victorov, 1978), which has increased by some 14 years since Beckman developed the USGS equations. Figure 2.3 plots the standard error of the skew coefficient against length of record. For the gages used in both this study and the Beckman study, the standard errors of skew are 0.388 and 0.472, respectively. Preliminary results by Hotchkiss and Cordes (1993) using the new regression equations show significant improvements in the LP3 estimates for those gages.

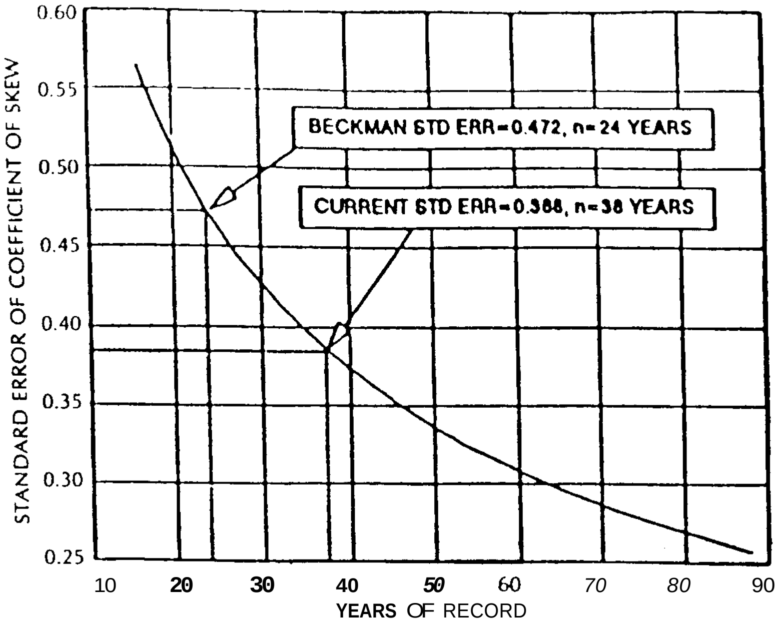


Figure 2.3 Standard error of skew coefficient plotted against length of record.

Chapter 3

PREVIOUS RESEARCH

As stated earlier, this project is the third in a series of studies funded by NDOK to analyze culvert and bridge design procedures. The first project, entitled "**A** Hydrologic Evaluation of Twenty-four Small Watersheds in Nebraska," was completed by Riley in 1988. The second project, entitled "Hydrologic and Hydraulic Design of Culverts," was completed by McCallum in 1992. This chapter highlights the results and recommendations of the previous work, and attempts to connect all three studies coherently.

RESULTS FROM RILEY

Riley's study (1988) evaluated two runoff models, the Rational Method and ***SCS*** TR-55. The curve numbers used in the TR-55 analysis were taken from a generalized curve number map instead of being determined in the manner set forth in the TR-55 manual (SCS, 1986). Twenty-four small, ungaged rural watersheds in Nebraska were chosen from NDOR culvert design projects. These sites ranged from 35 to 1300 acres in size.

For each runoff model, four different time-of-concentration methods were used. These four methods included a nomograph currently used by NDOR, the Kirpich equation, the SCS lag equation, and an estimate based on Manning's velocity. The results of the two runoff methods using each time of concentration method were compared at each of the **24** sites.

A detailed hydraulic analysis was also performed at four of the selected sites. These four sites were chosen because of the detailed data that were available. This analysis evaluated the effect of storage and flow routing through the culverts.

Riley used computer evaluation of different design methods and reviewed technical literature. No actual data were collected at any of the sites. He made the following recommendations:

1. Calculate times of concentration by summing overland flow times and channel flow times. Overland flow should be calculated by a technique that takes into account

the runoff potential of the basin. This implies using an equation with a runoff coefficient.

2. Use the TR-55 method for watersheds greater than 300 acres. This allows the watershed to be divided into homogeneous areas, and the peak discharges of each subarea to be routed to the basin outlet. Continue to apply the Rational Method to watersheds less than 300 acres.

3. When calculating watershed slope, use the Gray Method. In this method, a straight line is drawn in the profile of the watershed slope from the outlet, equally dividing the areas above and below the line.

4. Add a frequency coefficient to the rational method. This makes the runoff potential more representative of higher return period events.

5. Use the intensity-duration-frequency (IDF) curves developed in the Riley study instead of the IDF curves currently used by NDOR to determine rainfall intensity. The two sets of curves are based on dissimilar rainfall regions.

6. When using the IDF curves, examine a range of intensities bracketing the design duration.

7. Include a range of frequency events and evaluate potential storage.

Riley recommended further research on application of a risk perspective to culvert design. Because of the large amount of money the State spends on small watersheds, his judgment was that researching culvert design would be a wise investment. Other topics he suggested for future investigation were the relationship between runoff storage and reduction in headwater, and the effect of using generalized curve numbers in the **SCS** TR-55 method.

RESULTS FROM McCALLUM

McCallum's project (1992) expanded on some items addressed in the Riley study. The goal of this project was to determine the most applicable method for estimating peak discharges. This included determining the best method for obtaining time-of-concentration estimates. Data were collected and compared to results of different estimation methods for both time of concentration and peak discharge.

The first step in McCallum's research was to find suitable gaging sites. Four sites were chosen, all smaller than 1.8 square miles, and on agricultural land. Previously gaged sites were used so that peak discharge results could be compared to LP3 estimates. Stream gages were placed at the main site as well as on upstream culverts. This allowed for measurement of time-of-concentration and peak discharge on watersheds of several different sizes within each larger basin. Rain gages were placed at the centroid of each of the four basins. With the rain gages and the stream gages on the watersheds, both the time of concentration and the peak discharge could be physically measured for each significant rainfall event.

The next step was to analyze several different methods for estimating time of concentration and peak discharge. Seven time-of-concentration equations and eight peak discharge methods were evaluated. The results of these methods were then compared to the actual field data. One limitation of the field data was the lack of any high return period storms in the two years that the gages were in place.

Based on this research, McCallum made the following recommendations:

1. Continue to determine time of concentration by use of the NDOR nomograph.

2. Apply an adjustment factor of 1.5 along with the nomograph for agricultural watersheds.

3. To permit use on narrow, long watersheds, extend the length axis of the nomograph.

4. Use the Kirpich equation to estimate time of concentration if the nomograph **is** not applicable. Again, use the 1.5 factor for agricultural land. This will result in some over-design due to the use of a higher rainfall intensity and, consequently, a higher peak discharge. The higher rainfall intensity calculation is the result of slightly different variables in the computation process.

5. Continue the current NDOR peak discharge procedures until more data can be collected and the entire research project is completed. Specifically, the Rational Method should be used for watersheds of less than one square mile, and the Potter Method should be used for areas between 1 and *25* square miles. [Note: additional field data is currently being collected at the same study sites.]

6. **If** there were no basis for developing the new regression equations, then the design procedure should be changed as follows: use the USGS regression equations (Beckman Equation) for areas less than two square miles and continue to use the Potter Method for areas from **2** to *25* square miles. These methods require no time-of-concentration estimates. Once peak discharge research on larger watersheds is completed, replace the Potter Method with new methods that take this factor into account.

7. The IDF curves developed by Riley are better than the current NDOR IDF curves because they allow for longer storm duration.

8. Use the runoff coefficients from the Stephenson table for the Rational Method. The table of coefficients from the NDOR manual gave the best results, but proper selection of a C value is more likely with the Stephenson table. The latter includes additional factors such as more types of land use, corrections for slope, mean annual precipitation, and recurrence interval. This is shown by Table **2.4** in McCallum's report.

9. Data collection at the four sites should continue until a large event can be recorded. This should include only the main sites, using only the transducer gages, to allow faster data collection. **[As** stated above, this data collection has continued through the summer of 1993 and may continue beyond that.]

DISCUSSION

Riley and McCallum do not reach precisely the same conclusions. This is to be expected due to differing methods used by the two. Since McCallum's results are based on actual observations, in case of conflict (i.e., time-of-concentration calculations) his recommendations will be the ones incorporated into the new NDOR design manual.

Chapter 4

CURRENT DESIGN PROCEDURES

Currently, the NDOR Roadway Design Division and the NDOR Bridge Division each use different methods for determining peak discharges. Some disparites in design procedure are due to the sizes of drainage areas assigned to each division. Since the Roadway Design Division only designs culverts, their drainage areas are generally small. The Bridge Division, on the other hand, deals with relatively large drainage areas. This chapter covers procedures currently used by each division.

ROADWAY DESIGN DIVISION

The Roadway Design Division manual lists two basic methods for calculating peak discharge, the Rational Method and the Potter Method. Occasionally, however, other methods are used. These include the USGS regression equations, the SCS TR-55 method, and computer programs developed by NDOR. The following sections detail these methods.

Rational Method

The Rational Method is an empirical equation that is relatively simple to use. The classic form of the equation is shown below:

$$Q=CiA \tag{4.1}$$

where: Q = runoff (cfs)

C = dimensionless runoff coefficient

i = rainfall intensity (in/hr)

A = drainage area (acres)

Note that the units on the variables are not homogeneous. To convert from inches-acres/hour to cubic feet per second requires a coefficient of 1.008, which is close enough to 1.0 that it is generally ignored. Several assumptions and limitations are associated with the Rational Method (McCallum, 1992), which are listed below:

1. It assumes uniform rainfall over the entire watershed.
2. The peak discharge computed from the equation has the same frequency as the rainfall intensity (i) used in the equation.
3. The peak discharge occurs only while the entire watershed is contributing.
4. Conversely, the time-to-peak, or the time of concentration, is the time when the entire area is contributing.
5. The Rational Method does not account for runoff that is primarily channel flow.
6. Drainage areas must be small to ensure that the uniform rainfall assumption is met. The current NDOR manual calls for drainage areas to be less than or equal to 640 acres.
7. The runoff coefficient *C* is considered constant for each storm.

The value for C is obtained from tables based on land use or cover and surface slope. The table used in the current design manual is reproduced as Table 4.1.

Table **4.1** Values of Coefficient of Runoff (C) (from NDOR, 1984)

Surface Type	**0% - 2%**	**2% - 10%**	**Over 10%**
Pavement, Roof Surfaces, etc.	0.80	0.90	0.95
Earth Shoulder	0.55	0.60	0.70
Gravel or Stone Shoulders	0.45	0.50	0.60
Grass Shoulder	0.30	0.35	0.40
Side Slopes - Earth	0.50	0.60	0.70
Side Slopes –Turf	0.40	0.50	0.65
Median Strips - Turf	0.30	0.35	0.40
Dense Residential Areas	0.60	0.65	0.80
Suburban Areas with Small Yards	0.40	0.50	0.60
Cultivated Land - Clay and Loam	0.35	0.50	0.60
Cultivated Land - Sand and Gravel	0.25	0.30	0.35
Parks, Meadows and Pasture Land	0.20	0.25	0.35

Rainfall intensity is obtained in two steps. First, the time of concentration (t,) is calculated. Next, the intensity is obtained from an IDF curve. The time of concentration is determined **by** use of a nomograph as shown in Figure **4.1.**

To use the nomograph, first find the difference in elevation between the divide and the watershed outlet (H). Then measure the total flow length (L). These two values are generally obtained from a topographic map. Use a straight edge to connect the two points and continue the line to a point on the t_c **axis.** This value is the time-of-concentration (t,).

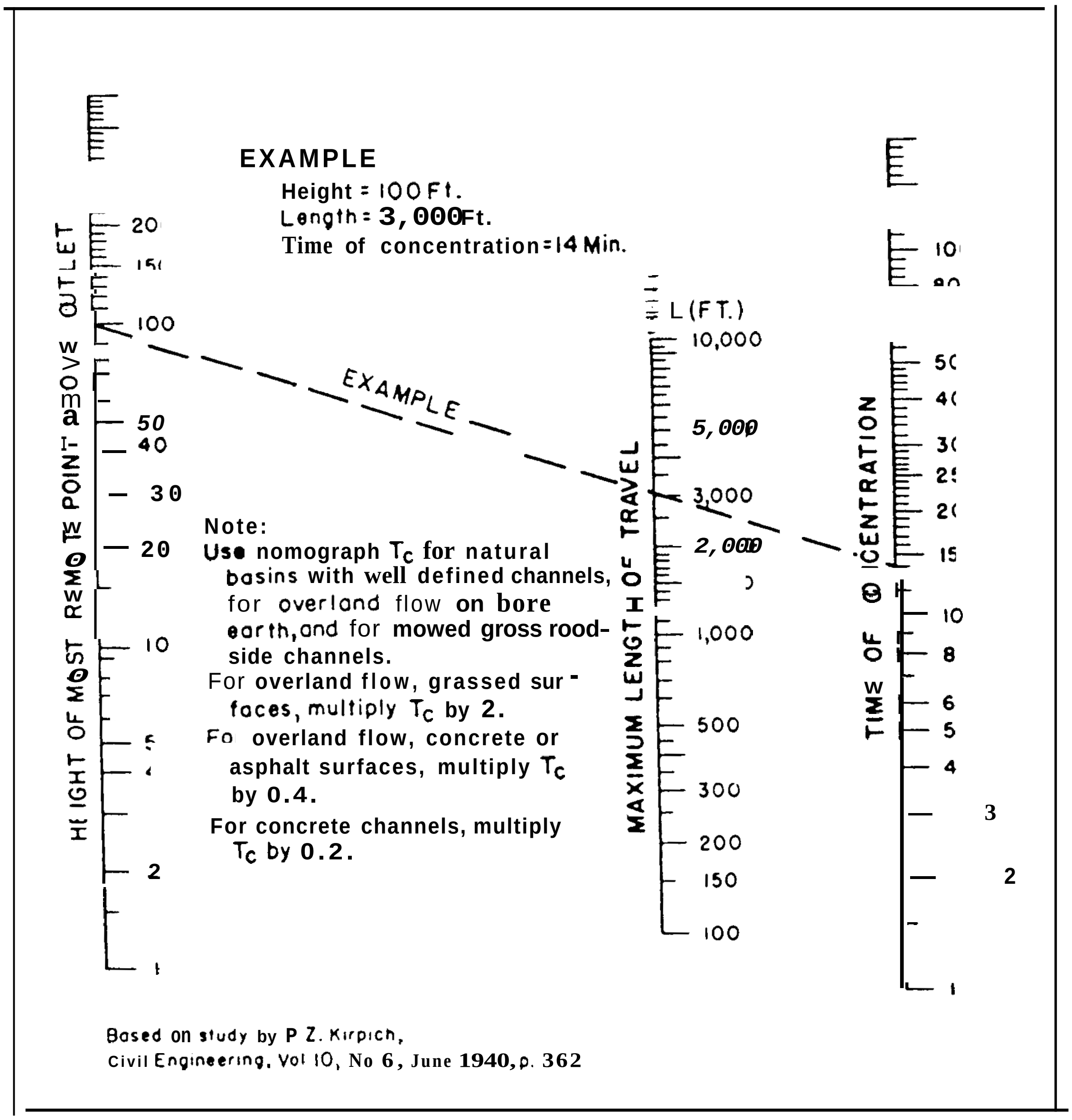


Figure 4.1. NDOR nomograph for calculating t_c (from NDOR, 1984)

Once t_c has been calculated, the IDF curves are used to determine intensity (i). NDOR currently divides Nebraska into eight different rainfall regions (Figure 4.2), each of which has an IDF curve. The IDF curve for Region 1 is shown in Figure **4.3** as an example. To obtain i, enter the IDF curve along the x-axis using t_c as the time. Next, follow this time vertically until it intersects the curve corresponding to the required return period. From the point of intersection, follow the line to the left to obtain i.

Once C and i have been determined, the only remaining variable is the area. This is measured directly from a topographic map using a planimeter. When all of the values have been determined, the calculation is straightforward.

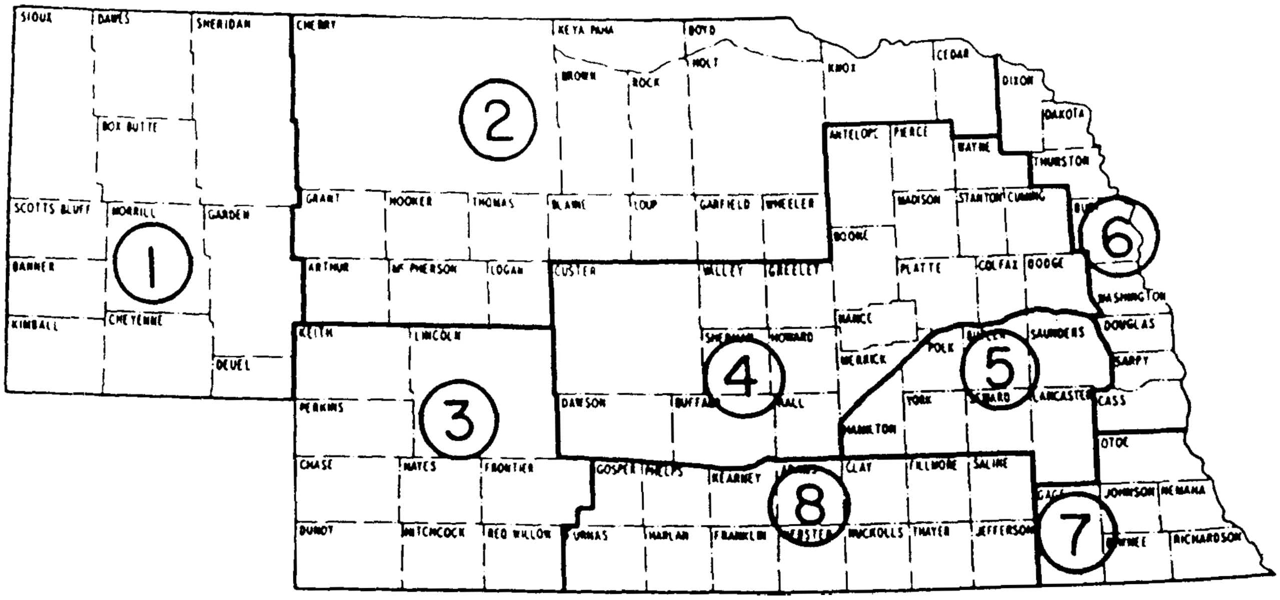


Figure 4.2. Rainfall intensity zones for Nebraska (from NDOR, 1984)

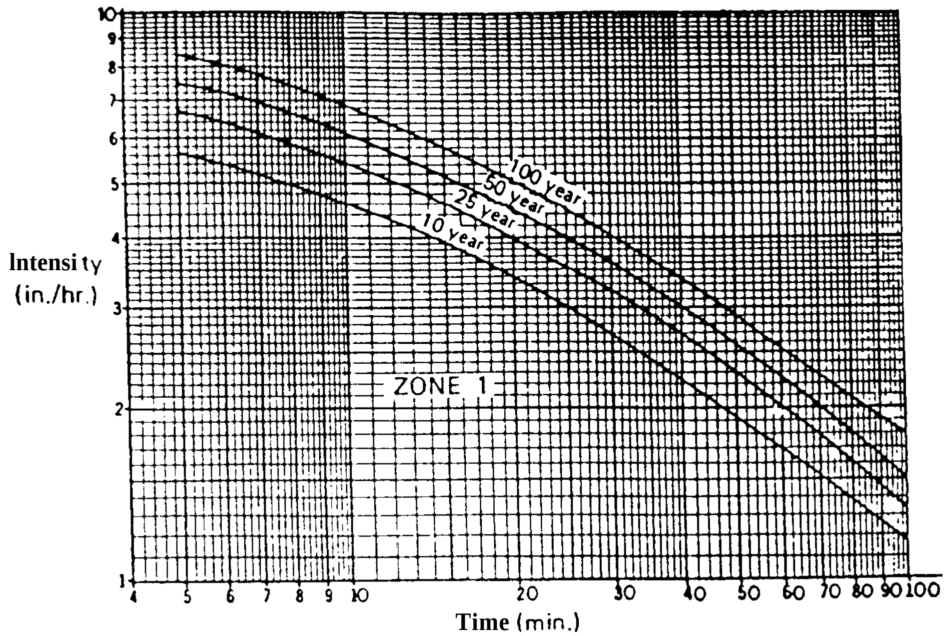


Figure 4.3. IDF curve for rainfall Zone 1 (from NDOR, 1984)

Potter Method

The Potter Method is used by NDOR for basins with drainage areas between 640 and 16,000 acres, except for the Sandhills region. It is a flood index method, using precipitation and topographic indices.

The first step in using the Potter Method is to obtain the drainage area, the channel length, and elevations at the headwater, at 0.7 the length of the channel, and at the outlet. These can be measured from a topographic map. Next, using these values, the topographic index **(T)** is determined using the following equation:

$$T = \frac{0.3L}{\dfrac{\sqrt{EL(HW)-EL(0.7L)}}{0.3L}} + \frac{0.7L}{\dfrac{\sqrt{EL(0.7L)-EL(O)}}{0.7L}} \tag{4.2}$$

where: **L** = length of channel (mi.)

EL(HW) = elevation at headwaters (ft.)

EL(0.7L) = elevation at 0.7 channel length (ft.)

EL(O) = elevation at outlet (ft.)

The precipitation index is determined from the map illustrated in Figure **4.4.** Values for the precipitation index are defined as the amount of precipitation in inches that might be exceeded during a 60-minute period once every 10 years, on the average.

The 10-year index flood is taken from the nomograph shown in Figure **4.5.** Enter the graph at the lower left-hand side with the drainage area. Then, proceed up until the line representing the previously computed topographic index is reached. From this point, proceed to the right until the line representing the precipitation index is reached, and then move up to the top to the graph to find the corresponding 10-year flood index.

The next step in the Potter Method is to determine the similarity of the design watershed to those used in the original calibration of the method. To do this, calculate the topographic index (T_e) of the calibration watersheds from the nomograph shown in Figure **4.6.** The percentage difference between T and T_e is then calculated using Equation **4.3:**

$$100\left(\frac{T_e - T}{T_e}\right) \tag{4.3}$$

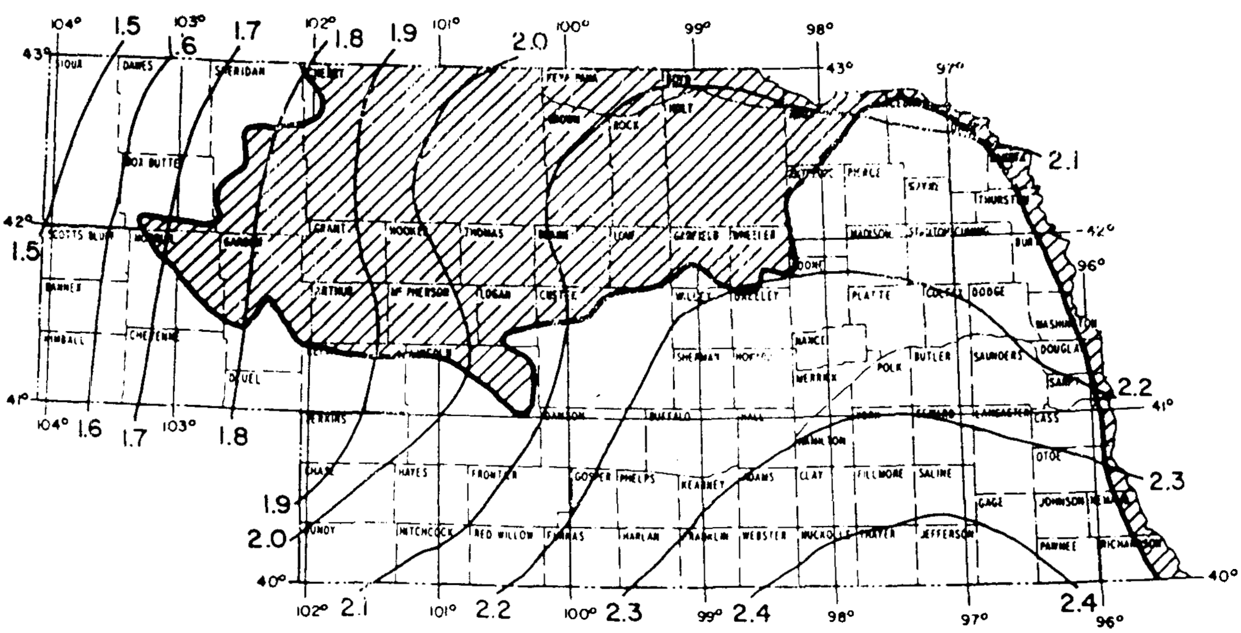


Figure **4.4.** Precipitation index **for** Nebraska **(from** NDOR, **1984)**

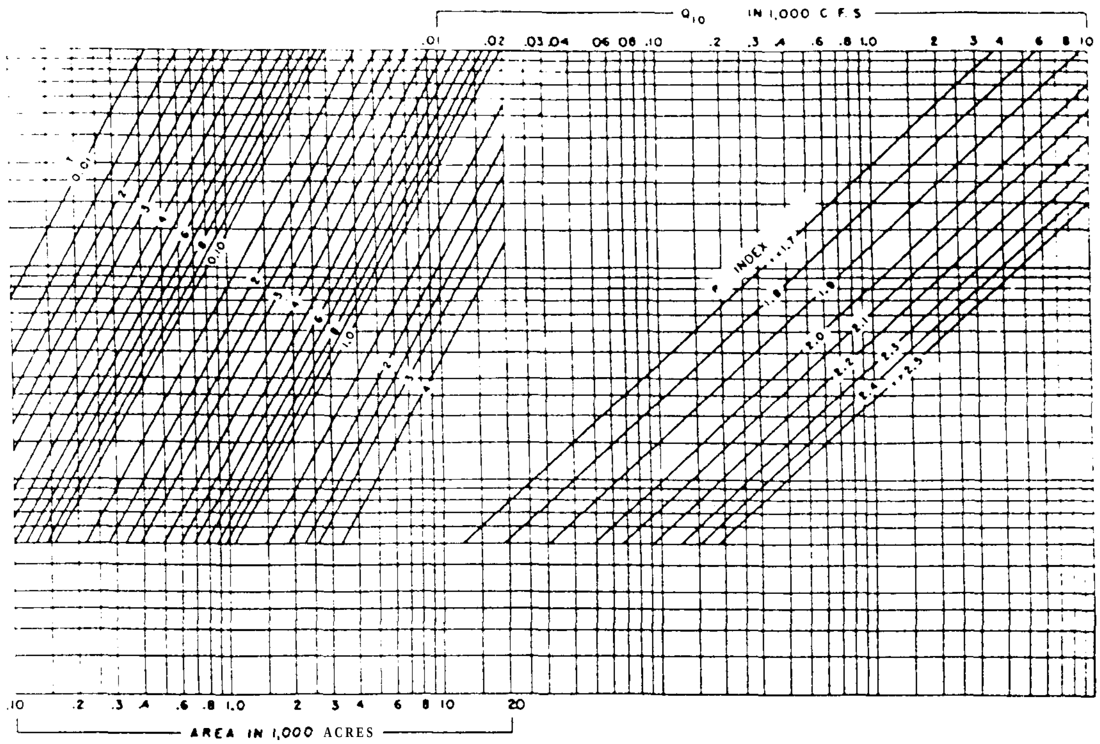


Figure 4.5. Nomograph for determining the 10-year flood index Q_{10} (from NDOR, 1984)

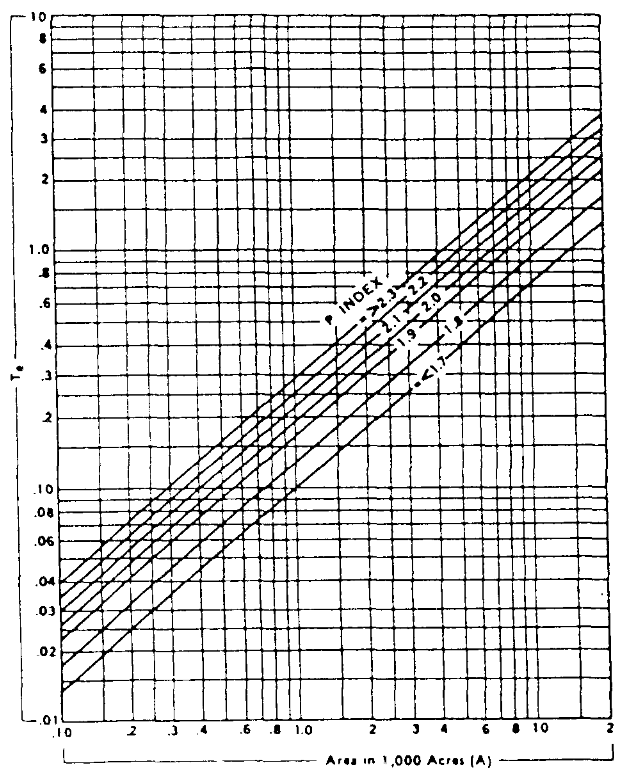


Figure 4.6. Nomograph for calculating the topographic index T_e (from NDOR, 1984)

If the difference is greater than +/- 30 percent, the watersheds are considered dissimilar and a correction factor must be used. This correction factor is obtained from the graph shown in Figure 4.7.

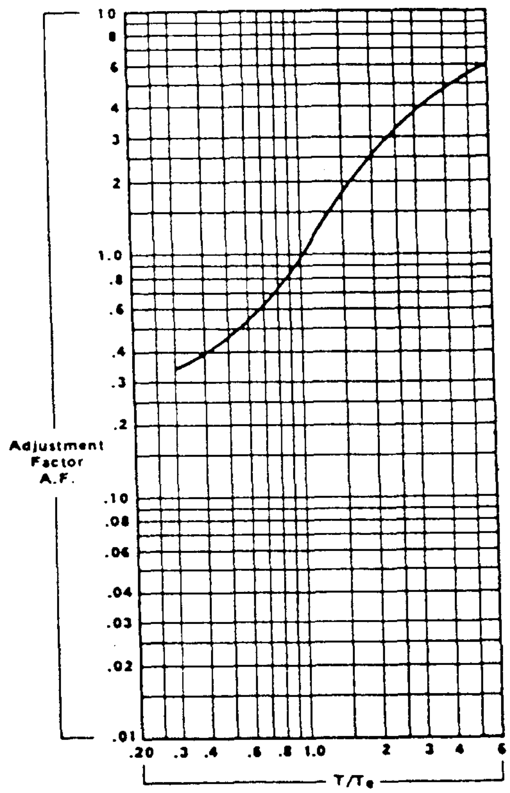


Figure 4.7. Adjustment factor for the Potter Method (from NDOR, 1984)

The final step in the Potter Method is to convert the 10-year discharge to the design discharge required. This is done by use of the following equations:

$$Q_2 = 0.605\ (Q_{10})$$
$$Q_5 = 0.813\ (Q_{10})$$
$$Q_{25} = 1.186\ (Q_{10})$$
$$Q_{50} = 1.384\ (Q_{10})$$
$$Q_{100} = 1.589\ (Q_{10})$$

QVALUES Computer Program

The Department of Roads has developed a computer program to aid in computing peak discharge. The program, QVALUES, was written by an NDOR engineer and is available on the main frame computer. This program is recognized in the current design manual as a way to determine peak discharges. It uses either the Rational Method or the Potter Method to compute these peak flows, depending on the drainage area input. The inputs to the program are the same as required for the methods previously discussed.

SCS TR-55 Method

The TR-55 method is not listed in the current design manual, but it is used by some NDOR engineers. It is available to them on a personal computer. TR-55 uses either the graphical or tabular method to estimate peak discharges. The former estimates only peak flow, and the latter generates a complete hydrograph. The graphical method is discussed below since this section deals only with estimating peak discharges and not with generating hydrographs.

The first step in using the TR-55 method is to determine a curve number (CN). The CN value is dependent upon land use, soil type, and hydrologic condition of the basin. CN values are obtained from charts (SCS, 1986). One assumption of this method is that the watershed is homogeneous, which means it can be represented by one CN value. **If** this is not the case, an area-weighted CN may be used.

The total runoff from the basin is calculated by the following formula:

$$Q=\frac{(P-0.2S)^2}{(P+0.8S)} \tag{4.4}$$

where: Q = total runoff (in.),

P = rainfall (in.),

S = potential maximum retention after runoff (in.),

and S is computed as follows:

$$S=\frac{1000}{CN}-10 \tag{4.5}$$

Once the total runoff is known, the peak flow rate is determined by the following equation:

$$q_p = q_u A_m Q F_p \tag{4.6}$$

where: q_p = peak discharge (cfs),
q_u = peak discharge/mi.^{2} per inch of runoff,
A_m = drainage area (mi.^{2}),
Q = total runoff (in.),
F_p = pond and swamp adjustment factor.

The computer program is user-friendly and quick to use. The user inputs include drainage area, CN, time of concentration, and rainfall depth and frequency. The rainfall depth can be obtained from a rainfall atlas. The program then provides the peak flow for each storm entered.

Beckman Regression Equations, WRI 76-109

The Beckman regression equations (USGS, 1976) are not specifically mentioned in the current design manual. However, some engineers in the Roadway Design Division do use these, either as a primary method or as a check of the values computed using other methods. This method is used more widely in the Bridge Division than it is in the Roadway Design Division. Therefore, it will be discussed in the next section.

BRIDGE DIVISION METHODS

The Bridge Division computes peak discharge estimates using several different methods, which are then compared. The choice of final design estimate is based upon the engineer's experience and judgement. Of the eight methods that can be used, five or six are used in each application, depending on data availability, drainage area, and location. These methods include Circular 458 (USGS, 1962), WSP 1679 (USGS, 1966), WSP 1680

(USGS, 1968), WRI 76-109 (USGS, 1976), the Rational Method, the Potter Method, NDOR Index Flood Method, and gaging station records. These methods are discussed below, except for the Rational and Potter Methods which were discussed in the previous section.

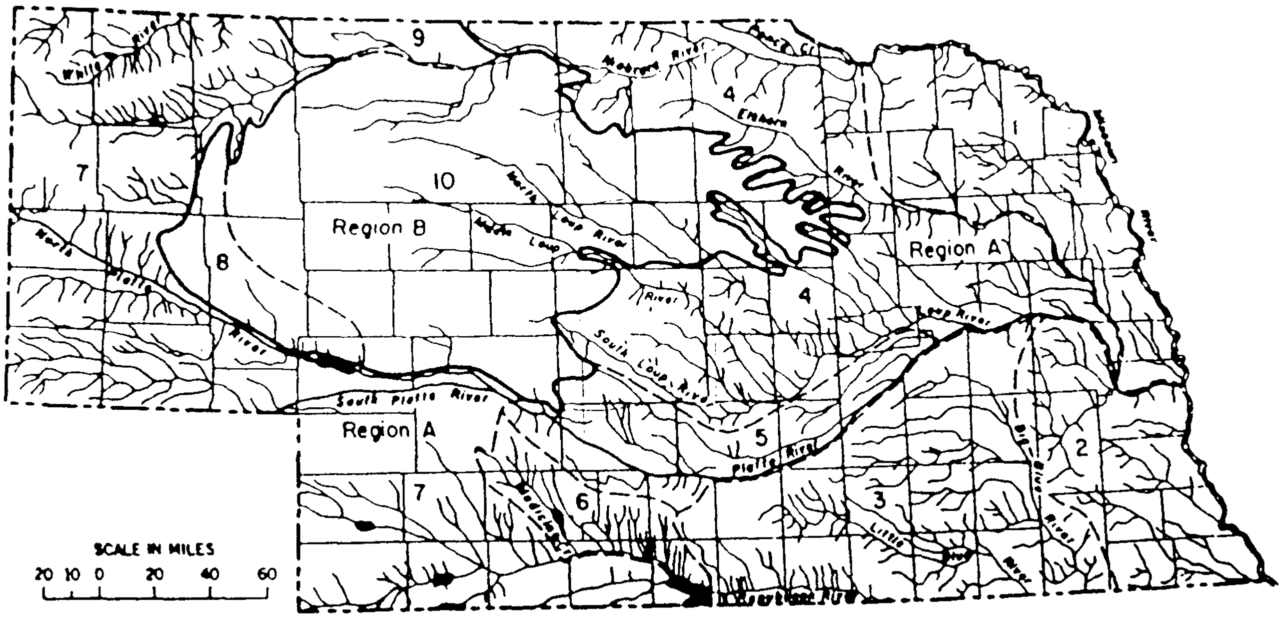


Figure 4.8. Map of Nebraska showing flood-frequency regions and hydrologic areas (USGS Circular 458).

Circular 458 Method

This method is applicable for drainage areas within Nebraska that are under 300 square miles. It was developed by analyzing the maximum peak flows for 142 gages in Nebraska. Based on these records, relationships for the mean annual flood were developed for 10 hydrologic areas, shown in Figure 4.8. These relationships are dependent only upon drainage area. The mean annual flood is defined as the 2.33-year flood.

The first step in using this method is to determine which of the 10 hydrologic areas is applicable to the design. Then the mean annual flood can be determined from the drainage area by using the appropriate equation, shown in Table 4.2. The mean annual flood is then related to the return period of interest by a simple ratio. Ratios for each area are shown in Table 4.3.

Table 4.2. NDOR Bridge Division hydrologic equations for finding mean annual flood (Q2.33) from drainage area (DA).

	Hydrologic Area	Equation for Discharge
REGION A	1	$\ln Q_{2.33} = 5.713 + 0.5271 * \ln(DA)$
	2	$\ln Q_{2.33} = 5.999 + 0.5511 * \ln(DA)$
	3, 4	$\ln Q_{2.33} = 3.634 + 0.6862 * \ln(DA)$
	5	$\ln Q_{2.33} = 3.806 + 0.4985 * \ln(DA)$
	6	$\ln Q_{2.33} = 4.972 + 0.5145 * \ln(DA)$
	7	$\ln Q_{2.33} = 2.265 + 0.8354 * \ln(DA)$
REGION B	8	$\ln Q_{2.33} = 2.369 + 0.7404 * \ln(DA)$
	9	$\ln Q_{2.33} = 1.645 + 0.7155 * \ln(DA)$
	10	$\ln Q_{2.33} = 3.134 + 0.7232 * \ln(DA)$

Table 4.3. Ratios of recurrence interval flood (QRI) to mean annual flood (Q2.33).

	Q_{RI}	$Q_{RI}/Q_{2.33}$
REGION A	Q_{10}	2.60
	Q_{25}	3.80
	Q_{50}	4.80
	Q_{100}	5.90
REGION B	Q_{10}	1.50
	Q_{25}	1.80
	Q_{50}	2.20
	Q_{100}	2.60

Water Supply Papers 1679 and 1680

The equations in WSP 1679 and WSP 1680 are similar to those of Circular 458, but were developed on a nationwide scale. WSP 1679 covers a region including watersheds that drain into the Missouri River above Sioux City, Iowa, in the extreme northern part of Nebraska. The region covered by WSP 1680 includes the rest of Nebraska (Figure 4.9).

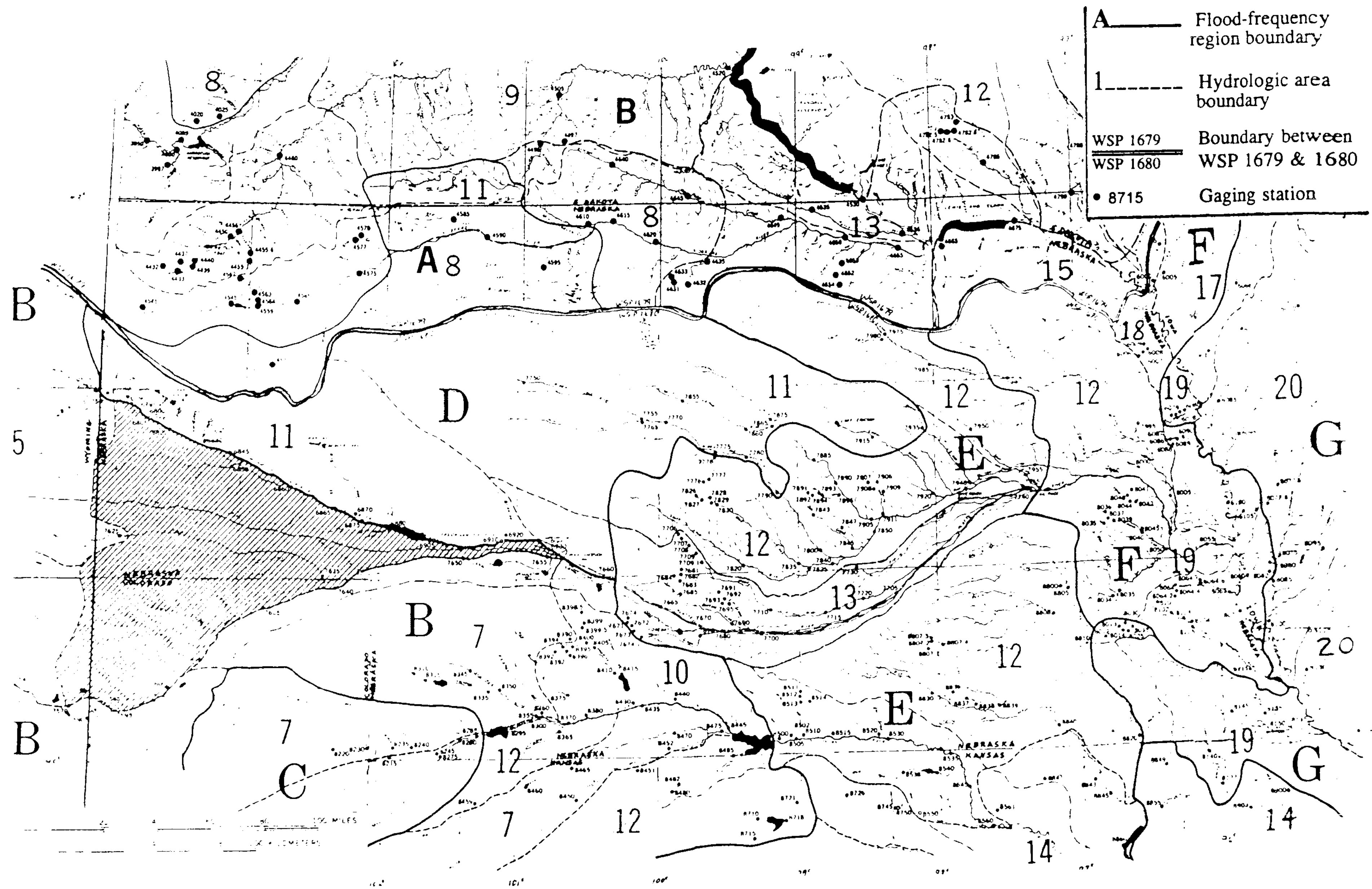


Figure 4.9 Nebraska hydrologic areas according to WSP 1679 and WSP 1680.

The two WSP methods for estimating peak discharge are mutually exclusive and depend on the location of the design. The equations and ratios that apply to Nebraska are listed in Tables 4.4 and 4.5.

Table 4.4. Equations for peak discharge estimation from WSP 1679 and WSP 1680.

Area	WSP 1679	WSP 1680
7		$\ln Q_{2.33} = 1.6769 + 0.7581 * \ln(DA)$
10		$\ln Q_{2.33} = 5.3239 + 0.4754 * \ln(DA)$
11	$\ln Q_{2.33} = 1.757 + 0.7150 * \ln(DA)$	$\ln Q_{2.33} = 3.3487 + 0.7108 * \ln(DA)$
12		$\ln Q_{2.33} = 4.3068 + 0.5516 * \ln(DA)$
13	$\ln Q_{2.33} = 3.621 + 0.6774 * \ln(DA)$	$\ln Q_{2.33} = 3.7136 + 0.4371 * \ln(DA)$
15	$\ln Q_{2.33} = 5.746 + 0.5172 * \ln(DA)$	
19		$\ln Q_{2.33} = 5.7429 + 0.5652 * \ln(DA)$

Table 4.5. Peak discharge values for Nebraska Regions A-G, developed from graphical representations in WSP 1679 and WSP 1680.

	WSP 1679		WSP 1680					
Interval \ Region	A	B	B	C	D	E	F	G
C_{25}	2.60	4.17	6.00	4.80	2.20	4.05	4.90	3.00
C_{50}	3.11	5.35	8.10	6.65	2.60	4.95	6.25	3.85
C_{100}	3.64	6.67	10.40	9.00	3.00	5.90	7.70	4.80

Beckman Regression Equations, WRI 76-109

This method uses regression equations developed for Nebraska in 1976 by regressing basin characteristics against peak flow estimates for different return periods. The peak flow estimates were obtained by performing an LP3 analysis on stream flow records. The State was divided into five hydrologic regions, and regression equations were developed for each region. Each region uses two physical characteristics and one climatological characteristic as the variables in the equation. The equations follow the form shown earlier in Table 2.2. WRI 76-109 also contains maps which can be used to determine any of the climatological variables. The length, slope, and area characteristics can be measured from a USGS topographic map.

NDOR Index Flood Method

NDOR personnel developed this IFM in 1972 by performing a stepwise regression analysis of all stream gage records in Nebraska. This method involves calculation of a topographic index, a precipitation index, and a flood index. Estimate of the 50-year discharge is made, from which the 100-year flood is predicted. An outline of this method follows.

1. Measure from a USGS topographic map the following characteristics: drainage area (A, $mi.^2$), basin length (L_b, mi.), basin width (W_b, mi.), stream valley length (L_v, mi.), elevation at the rim (E_r), elevation at a control point (L,) located at 0.7 L,, and elevation at the outlet.

2. Calculate the average valley slope by the following equation, using the values obtained in step 1:

$$S_v=[\frac{(E_r-E_c)}{0.3L_v}+\frac{4(E_c-E_s)}{0.7L_v}]*0.2 \tag{4.7}$$

3. Calculate the topographic index (T_i) using the following equation:

$$T_i=A^{0.5}*(\frac{W_b}{L_b})^{1/3}*(S_v)^{0.5} \tag{4.8}$$

4. Determine the precipitation index (P_i), defined as the 12-hour, 50-year precipitation at the site divided by 5.

5. Obtain the runoff ratio (RR), which is the inverse of the drainage basin permeability. NDOR has soil maps for the entire state, with permeability rates **for**

each soil type calculated. The design permeability is determined by an area-weighted average permeability for the entire basin.

6. Use the following equation to find the flood index (FI):

$$FI = T_i * P_i * RR \tag{4.9}$$

7. Calculate the 50-year discharge (Q_{50}) as follows:

$$Q_{50} = 95{,}000(FI)^{2.15} \tag{4.10}$$

8. Taking Q_{50} times 1.25 results in a figure for the 100-year flood (Q_{100}):

$$Q_{100} = 1.25 * Q_{50} \tag{4.11}$$

Gage Records

The Bridge Division may also use gage records to calculate peak discharge, but only if gage records exist at or near the site. Such records can then be used to perform an LP3 analysis at the site. This method is described in further detail in the next chapter.

Chapter 5

USGS REGRESSION EQUATION UPDATE

The Nebraska office of the USGS performed a regional flow frequency analysis for the State (Beckman, 1976) which **used** gaging station records through water year 1972. Since that report was completed, an additional 19 years of stream gaging data have gone on record (the current study includes records through water year 1991.) There have also been new, standardized techniques developed for performing regional flow frequency analysis since the Beckman report was completed.

Based upon the new standardized techniques presented in Bulletin 17B (U.S. Water Resources Council, 1981), there are three basic steps to developing regional peak flow equations. The first step is to update the peak flow estimates at all gaging stations using LP3 analysis. These estimates will be used later in the regression equation development as dependent variables. In updating these peak flow estimates, a generalized skew term is used, which requires the development of a generalized skew map. This generalized skew is weighted with the station skew at each gage to eliminate the effect of extreme events. The three-step procedure is discussed in detail below.

Once these steps have been completed, regression equations can be developed. The regression equations use several stream flow characteristics and climatological variables to predict peak flows at the gaging stations. The results of these calculations can then be used to predict peak flows at locations where no gaging station records are available. These three steps are discussed in detail below.

LOG PEARSON TYPE III (LP3) UPDATE

The LP3 method is a statistical distribution, as discussed in Chapter *2* of this report. This distribution is recommended by the U.S. Water Resources Council in Bulletin 17B for determining flood flow frequencies. The LP3 distribution has three parameters: the mean, the standard deviation, and the skew coefficient of a data set. The data, in this case, are the annual peak discharges at **a** gaging station. The general equation for this distribution is:

$$Q_L = \overline{X} + KS \tag{5.1}$$

where: Q_L =logarithm of annual peak discharges,

X =mean of logarithms of annual peak flows,

K =factor dependent on skew and exceedence probability,

S =standard deviation of logarithms of annual peak flows.

The mean, standard deviation, and skew coefficients are calculated as below:

$$\overline{X}=\frac{\Sigma(X)}{N} \tag{5.2}$$

$$S=[\frac{(X-\overline{X})}{(N-1)}]^{0.5} \tag{5.3}$$

$$G=\frac{N*\Sigma(X-\overline{X})^3}{(N-1)(N-2)S^3} \tag{5.4}$$

where: N = number of items in data set,

X = logarithm of annual peak flow,

G = skew coefficient of logarithms of annual peak flows.

Figure 5.1 shows the relationship between the skew coefficient, the return period, and the frequency factor K.

To perform the LP3 analysis for this report, the computer program HECWRC was used. This program accompanies WRC Bulletin 17B. The data used for the update was obtained from the Nebraska office of the USGS. This data was then formatted for compatibility with the computer program.

After the records were obtained from the USGS and put in the proper format, the gages that did not fit the cirteria prescribed by Bulletin 17B had to be eliminated. Only data

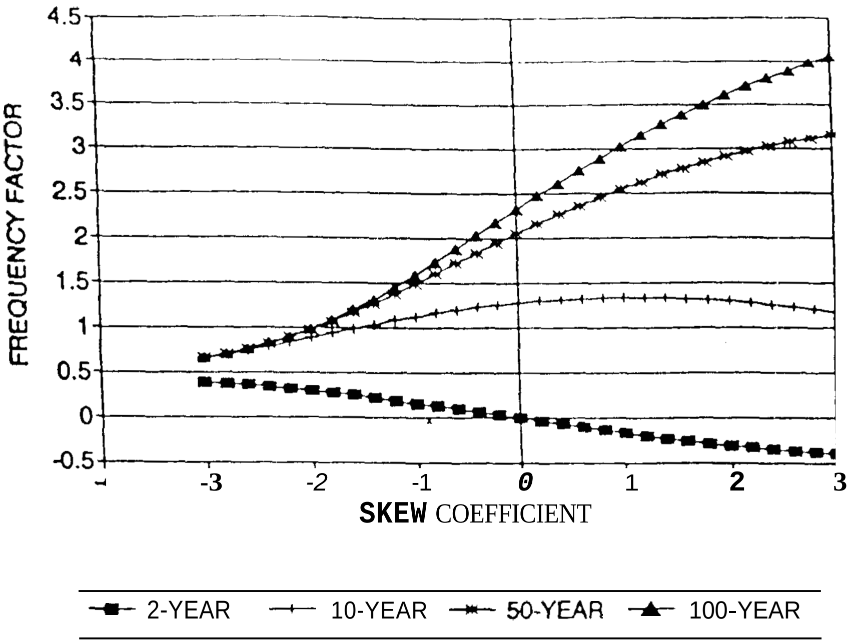


Figure **5.1** K-values for different skews **and** return **periods.**

from gages having peak discharge records for at least 10 years and nonzero peak flows for at least 75 percent of the records passed the first test for inclusion.

The next step was to determine which gages were on streams that were essentially uncontrolled. A list of all the dams in Nebraska was obtained from the Department of Water Resources. The gaging stations and their drainage basins were marked on one-degree-by-two-degree quad maps that included all of Nebraska and parts of Wyoming, South Dakota, Colorado, and Kansas. The dams were also located on the maps, and if more than 25 percent of the drainage area appeared to be controlled, the gage was eliminated.

SKEW MAP DEVELOPMENT

After selecting the gage sites that conformed to the specifications of Bulletin 17B, we determined the generalized skew coefficient. Bulletin 17B suggests the use of a generalized skew coefficient to be weighted with the station skew in order to eliminate the effect of extreme events. As Equation 5.4 shows, very small or very large values for X result in large: positive or negative values for the skew because $(X - \bar{X})$ is cubed. The effect of extreme events on small samples is especially pronounced.

Bulletin 17B contains a generalized skew map for the entire nation. This map, however, uses only gaging stations with records through 1973. The author of Bulletin 17B notes that this map may not be accurate for some regions, and recommends that users perform their own detailed studies for generalized skew relationships. Therefore, this study developed a skew using procedures outlined in the bulletin, as detailed below.

The first step was to determine which stations to use in developing the map. These stations had to meet the same criteria as the gages used in the LP3 update, with the additional requirement of 25 years of gage records instead of 10.

After the skew coefficient for the selected gages was calculated, the next step was to locate the centroid, or center of mass, of each of the drainage basins. This was accomplished by tracing the drainage areas and cutting out the shapes. **A** hole in the shape was then made with a pin, the pin was held horizontally, and the shape was allowed to pivot on the pin. **A** vertical line down from the pinhole was drawn on the shape. The pin was then moved to another location on the shape, and the procedure was repeated. The intersection of the two lines defines the centroid of the shape. The traced shape was then placed back over the map, and the centroid was transferred to the map. The Natural Resources Commission supplied the map that was used for most of the centroid locations. The few drainage basins corresponding to USGS gaging station locations not located on this map were traced from the one-degree-by-two-degree quad map mentioned above. After all of the centroids were located on one of the maps, the latitude and longitude for each gage was ascertained.

The next step was to average the skew coefficients. This was done in the same way that the skew map was developed for Bulletin 17B. The State map was divided into one-degree square quads, and all gage centroids falling within each quad were averaged. This average was then plotted at the center of that quad. The computer software package SURFER took these points and developed an isoline skew map. The X and Y coordinates of each point on the map were the coordinates of the center of each one-degree quad, and the Z coordinate was the quad's average skew coefficient. For this skew map, 143 gages were used, compared to 46 for the map developed in Bulletin 17B. Figure 5.2 shows the new skew map, and Figure 5.3 shows the approximate map taken from Bulletin 17B.

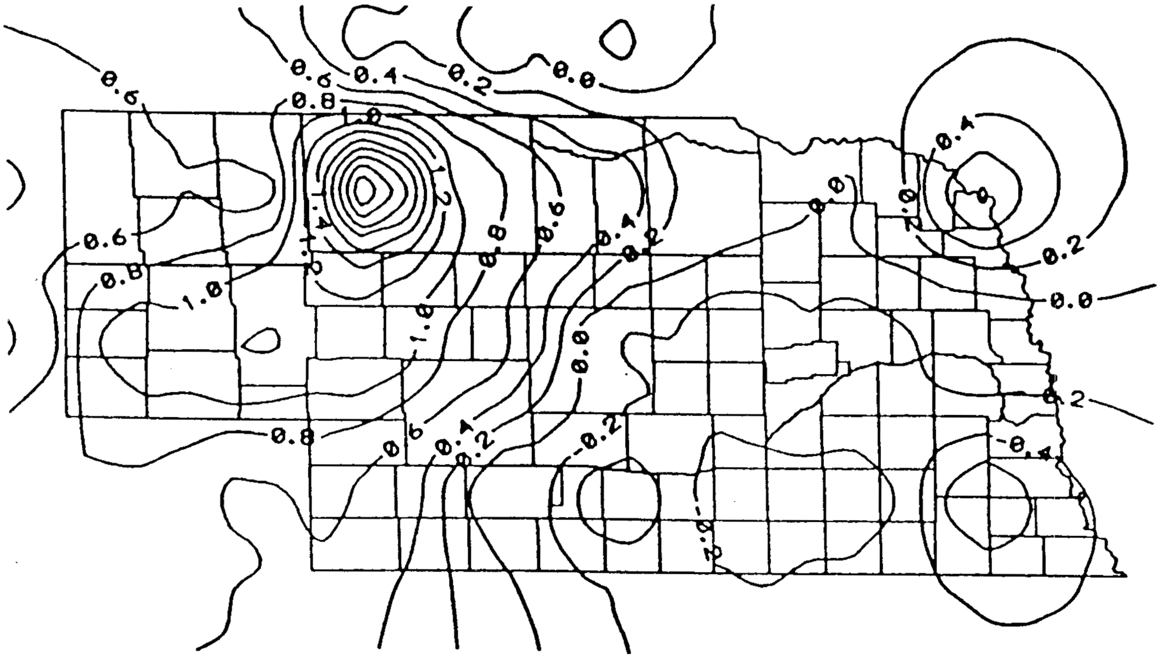


Figure 5.2 Updated skew map for Nebraska

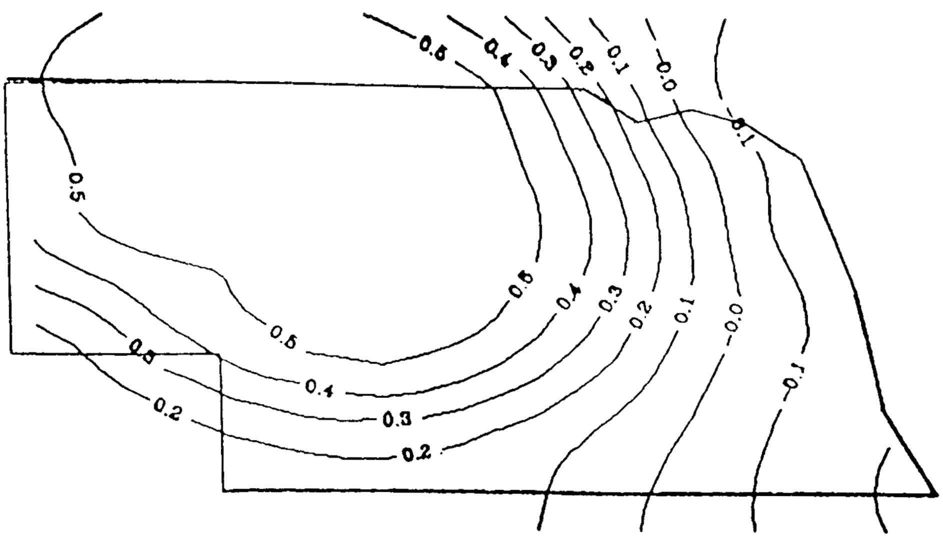


Figure 5.3. Bulletin 17B skew map (from Water Resources Council, 1981)

Bulletin 17B publishes a nationwide standard deviation of about 0.55 for station skews from its isolines. For Nebraska, however, this was found to be about **0.78.** The new skew map reduced standard deviations for Nebraska to about 0.59.

REGRESSION EQUATION DEVELOPMENT

The purpose of regression equations is to estimate peak flows at locations were gaging station records are not available. These equations were developed by using physical and climatological characteristics of the watersheds corresponding to each gaging station location. These characteristics were used as the independent variables, and the peak flows estimated from the LP3 distribution were used as the dependent variables. By measuring these characteristics at other locations, the peak discharges can be estimated.

The regression equation development process began by determining the physical watershed and climatological variables for each gaging station. The data for most gaging stations were gathered from the USGS database. Characteristics of missing stations were obtained from maps. The characteristics used are listed below:

A = Drainage area (mi.^{2}),

A_c = Contributing drainage area (mi.^{2}),

L = Length from station to basin divide along main channel (mi.),

S = Slope, measured from the elevations at .10 and .85 of the channel length, divided by L (ft./mi.),

P = Average annual precipitation (in.) [Figure 5.41,

$I_{24,2}$ = Rainfall intensity for a two-year, 24-hour event (in./hr.) [Figure 5.5],

$I_{24,50}$ = Rainfall intensity as above, except for a 50-year event (Figure 5.6),

$SN10$ = Equivalent moisture content of snow (in.) as of March 15 (Figure 5.7),

T_1 = Mean minimum January temperature (^{o}F) [Figure 5.8.],

T_2 = Mean maximum July temperature (^{o}F) [Figure 5.91,

T_3 = Normal daily maximum March temperature (^{o}F) [Figure 5.10],

E = Average annual lake evaporation (in.) [Figure 5.11].

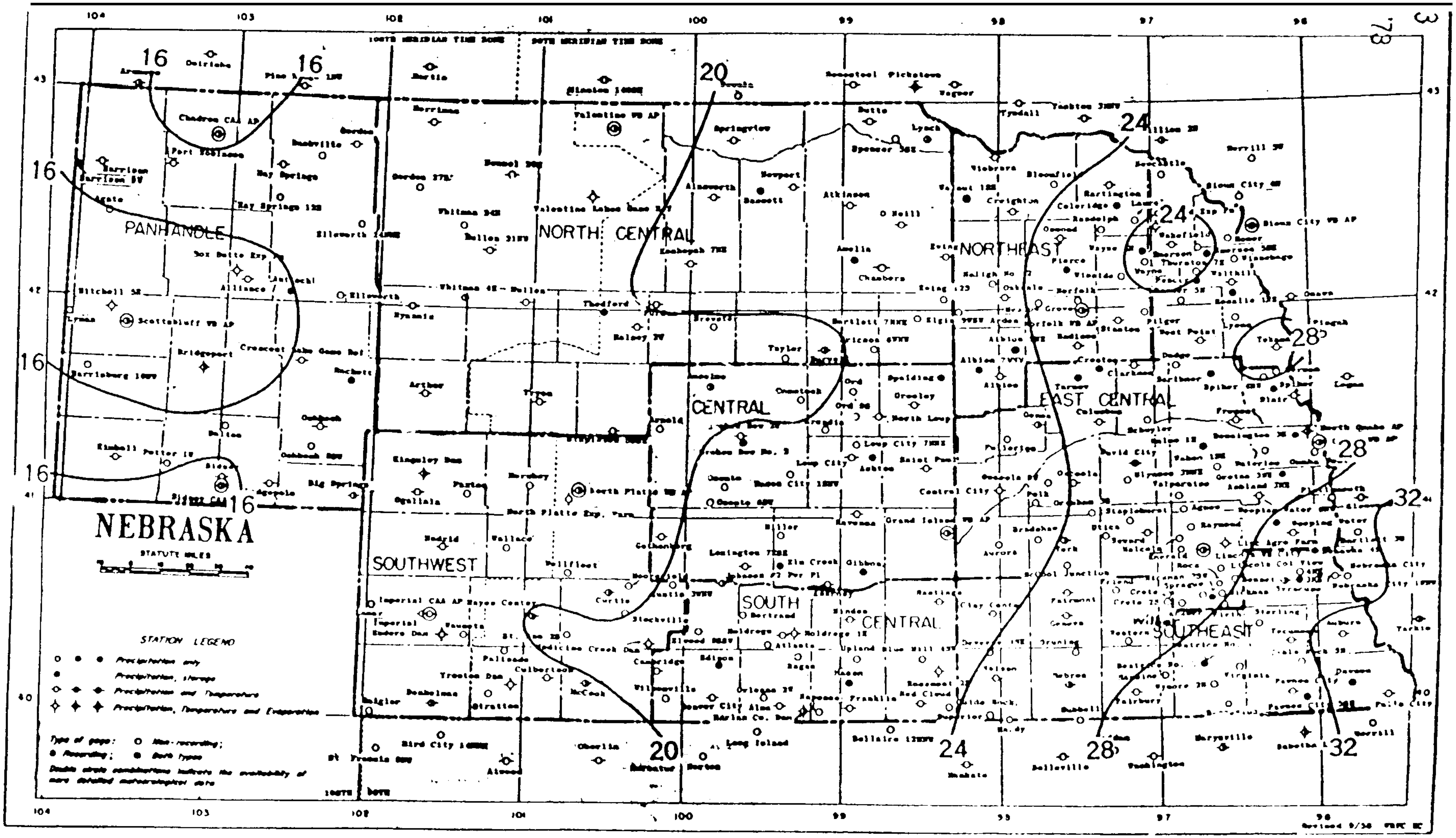
After all of these characteristics were determined for each station, the stations were divided into hydrologic regions previously defined in the USGS report (Beckman, 1976). These regions were shown earlier in this report (Figure *2.2).*

The regression process was then performed for each return period of interest and for each region using the statistical computer program **SAS.** Model selection was based upon the three-variable model resulting in the highest R^2, restricted to the same characteristics for each return period in a given region. Each return period, however, resulted in different best models. To handle this problem, the ten best models were considered for each return period in each region. They were then ranked according to the R^2 value, 1 being the lowest and 10 the highest. This was done for each return period. The rank values were added together for the region, and the model with the highest score was chosen to represent the region.

Problems were encountered in Region 1, however, using this method. Only two of the three variables were found to be significant in the model. For this reason, a stepwise regression method was performed in an attempt to build the best three-variable model, instead of basing it on the R^2 criteria. This procedure also resulted in only two variables being significant. For this reason, Region 1 is the only two-parameter model.

Region 1 had the poorest fit of all of the regions, a result which was also found in the USGS study (Beckman, 1976). This may be explained by the way Beckman delineated the regions. They were determined by plotting the residuals from the regression on a map. From this plot, the gages were divided into regions 2 through 5. The remaining gages that didn't fit well into any other region were lumped together in Region 1.

In the development of these equations, some cases were found to exert a high degree of influence on the regression line. The gage records in those cases were checked to be sure that no abnormalities were in the data, such as variables that fell far outside the usual range. Based on these examinations, there was no apparent justification to eliminate any of the data. The resulting equations from the regression update are shown in Table 5.1.



Based on period 1931-55

Figure 5.4. Mean annual precipitation (in.) (U.S. Weather Bureau 1959)

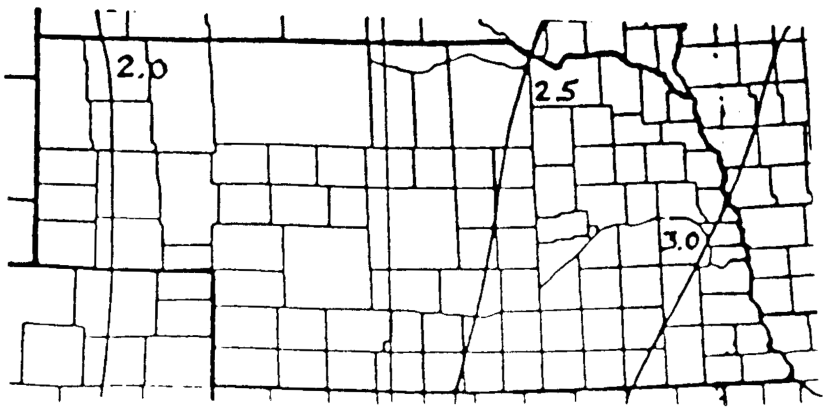


Figure 5.5. 2-year, 24-hour rainfall intensity (in.) (U.S. Weather Bureau, 1961)

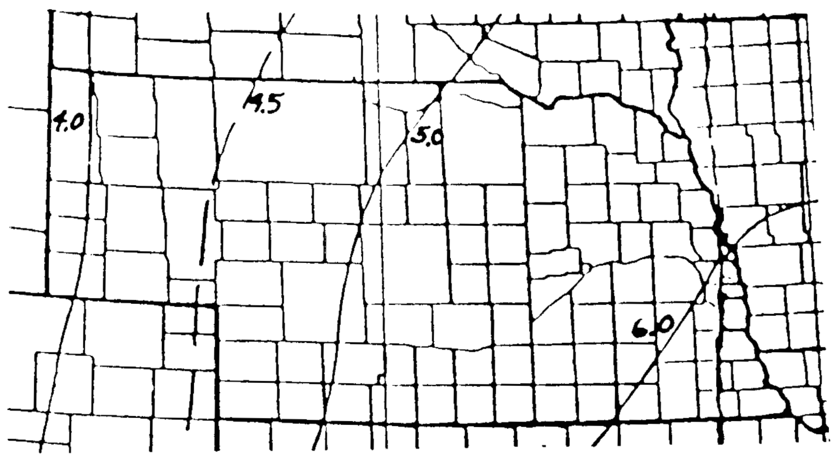


Figure 5.6. 50-year, 24-hour rainfall intensity (in.) (U.S. Weather Bureau, 1961)

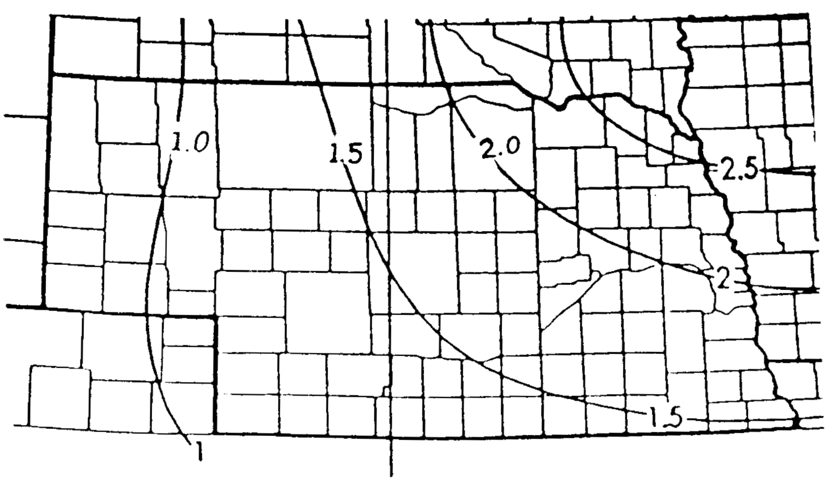
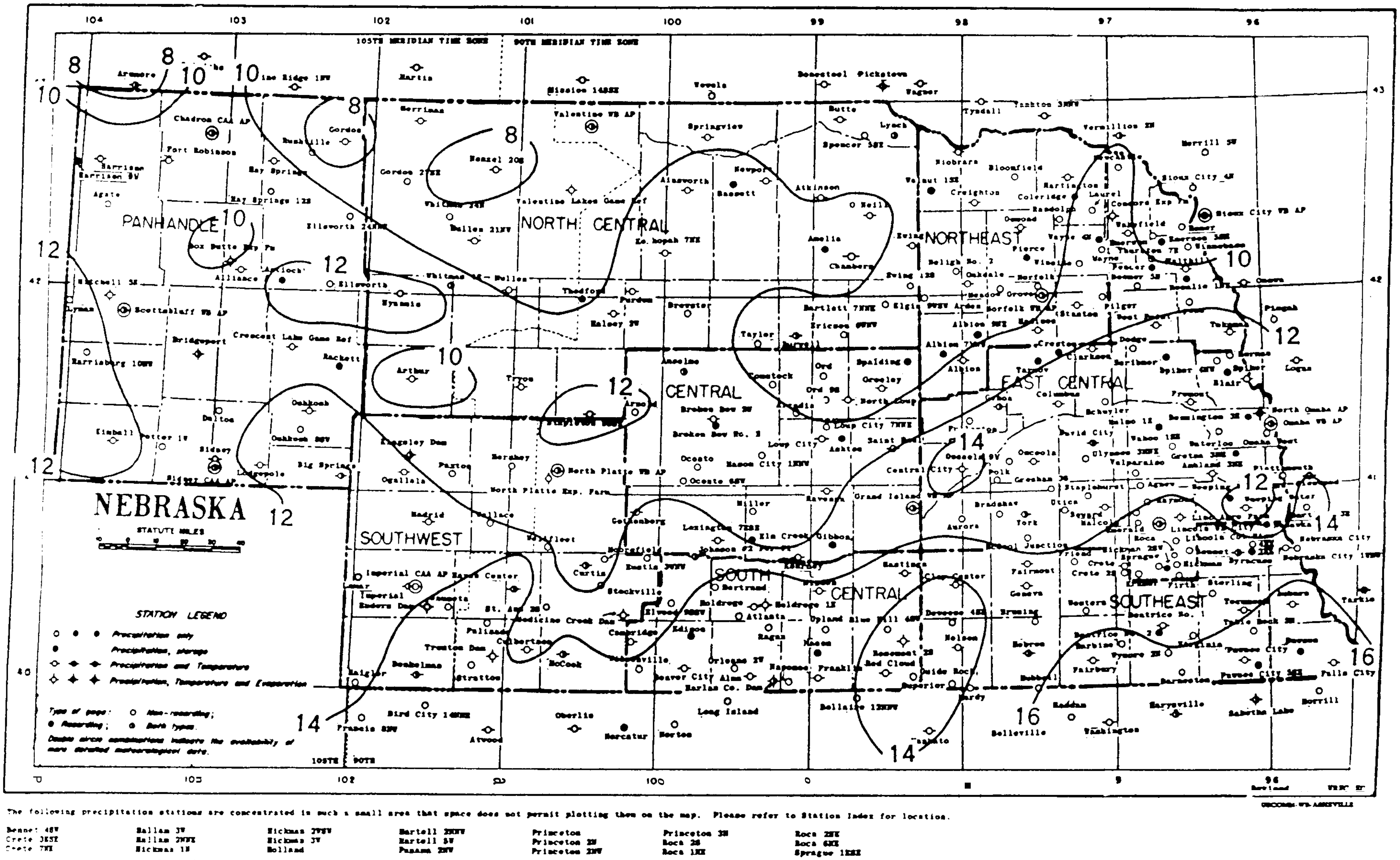
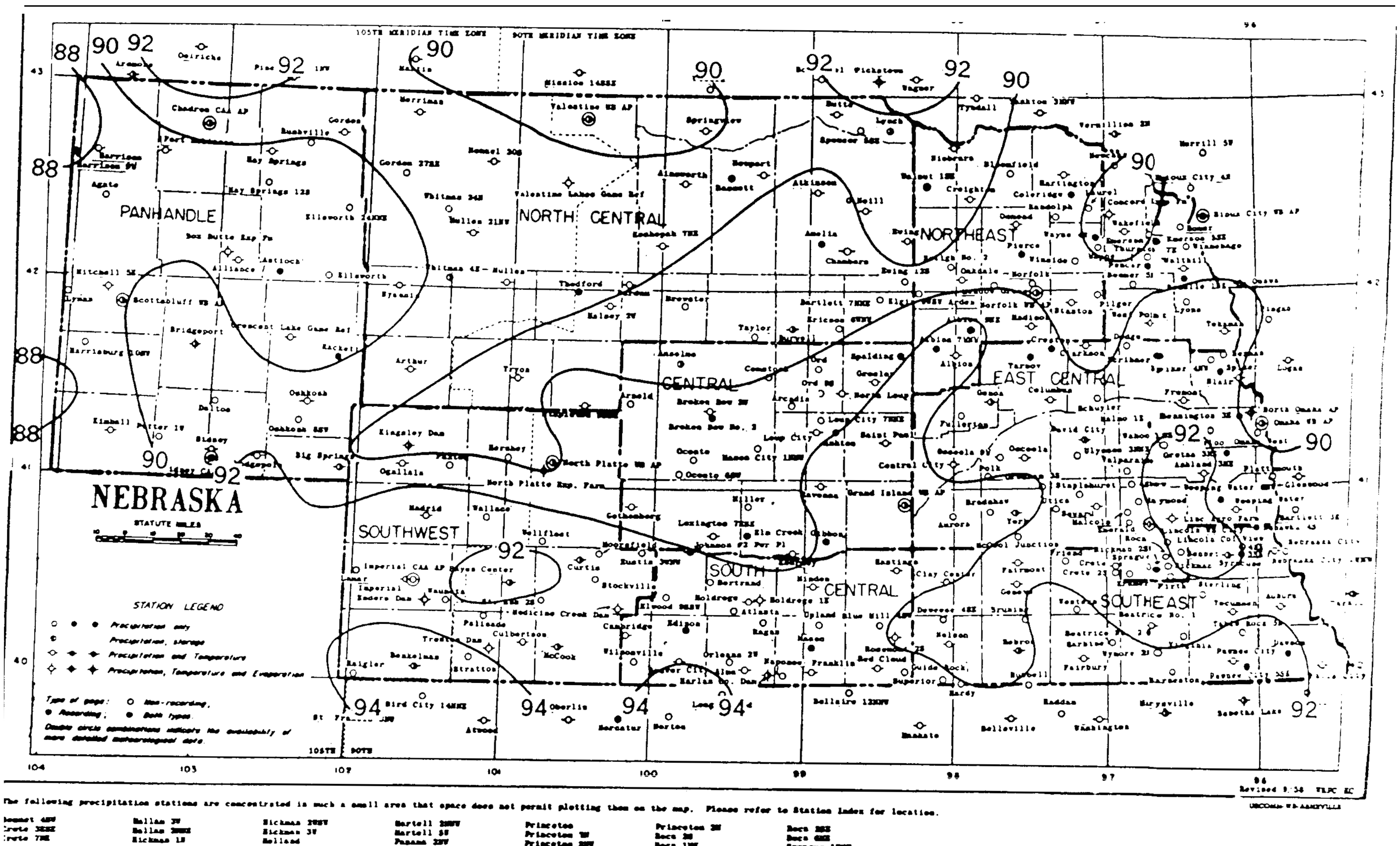


Figure 5.7. 10%-probability-equivalent moisture content of snow as of March 15 (in.) (U.S. Weather Bureau, 1964)



Based on period 1931-52

Figure 5.8. Mean minimum January temperature (°F) (U.S. Weather Bureau, 1959)



Based on period 1931-52

Figure 5.9. Mean maximum July temperature (°F) (U.S. Weather Bureau, 1959)

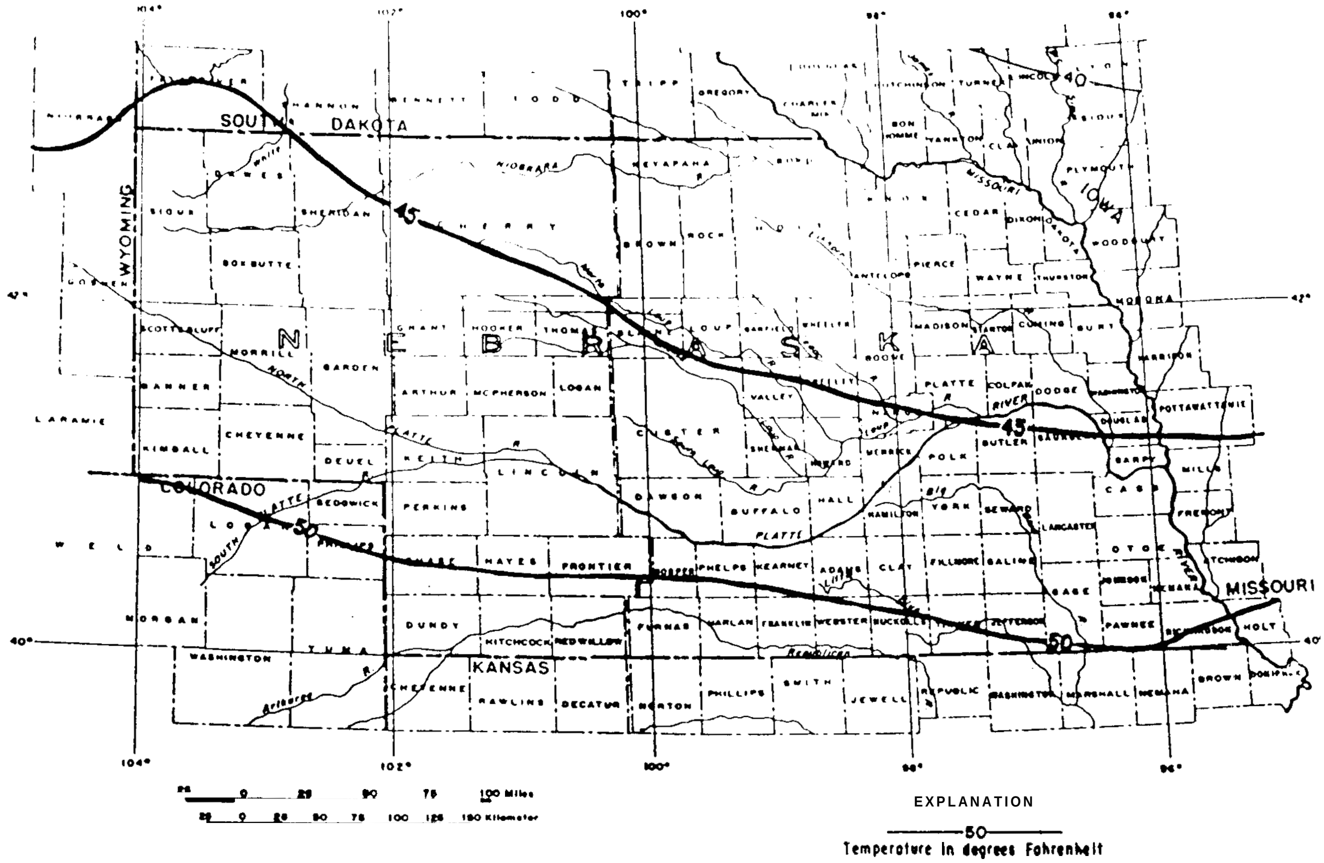


Figure 5.10. Normal daily March temperature (°F) (U.S. Weather Bureau, 1959)

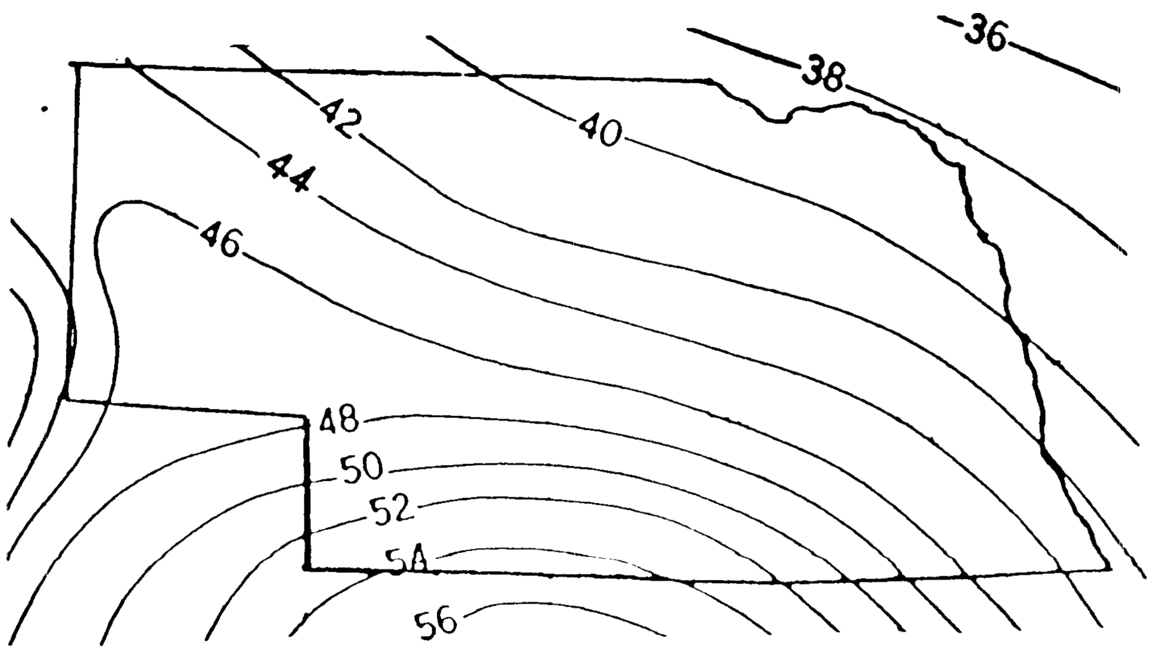


Figure 5.11. Average annual lake evaporation (in.) (U.S. Weather Bureau, 1959)

Table 5.1 New regional regression equations for Nebraska

Return Period	REGION 1	REGION 2
2	$Q_2 = 1.965\ A_c^{0.493}\ (P-13)^{1.44}$	$Q_2 = 0.269\ A_c^{0.912}\ S^{0.967}\ SN10^{2.337}$
10	$Q_{10} = 211.7\ A_c^{0.324}\ (P-13)^{0.314}$	$Q_{10} = 0.109\ A_c^{0.9917}\ S^{1.653}\ SN10^{2.607}$
SO	$Q_{50} = 6366\ A_c^{0.211}\ (P-13)^{-0.630}$	$Q_{50} = 0.0845\ A_c^{1.036}\ S^{2.005}\ SN10^{2.632}$
100	$Q_{100} = 23553\ A_c^{0.170}\ (P-13)^{-1.011}$	$Q_{100} = 0.0816\ A_c^{1.051}\ S^{2.119}\ SN10^{2.615}$
200	$Q_{200} = 82183\ A_c^{0.131}\ (P-13)^{-1.382}$	$Q_{200} = 0.0816\ A_c^{1.064}\ S^{2.216}\ SN10^{2.587}$
500	$Q_{500} = 400713\ A_c^{0.082}\ (P-13)^{-1.863}$	$Q_{500} = 0.0844\ A_c^{1.079}\ S^{2.326}\ SN10^{2.536}$
	REGION 3	REGION 4
2	$Q_2 = 7.57*10^{-10}\ A_c^{0.815}\ S^{0.599}\ P^{7.099}$	$Q_2 = 341.4\ A_c^{0.443}\ L^{0.126}\ (T_3-43)^{-2.062}$
10	$Q_{10} = 2.55*10^{-8}\ A_c^{0.722}\ S^{0.505}\ P^{6.657}$	$Q_{10} = 4741\ A_c^{0.914}\ L^{-0.783}\ (T_3-43)^{-1.960}$
50	$Q_{50} = 8.19*10^{-7}\ A_c^{0.688}\ S^{0.492}\ P^{5.908}$	$Q_{50} = 19516\ A_c^{1.285}\ L^{-1.411}\ (T_3-43)^{-1.903}$
100	$Q_{100} = 3.26*10^{-6}\ A_c^{0.681}\ S^{0.497}\ P^{5.581}$	$Q_{100} = 31008\ A_c^{1.433}\ L^{-1.648}\ (T_3-43)^{-1.876}$
200	$Q_{200} = 1.37*10^{-5}\ A_c^{0.677}\ S^{0.504}\ P^{5.226}$	$Q_{200} = 46677\ A_c^{1.573}\ L^{-1.871}\ (T_3-43)^{-1.850}$
500	$Q_{500} = 9.20*10^{-5}\ A_c^{0..673}\ S^{0.516}\ P^{4.740}$	$Q_{500} = 75811\ A_c^{1.752}\ L^{-2.148}\ (T_3-43)^{-1.819}$
	REGION 5	
2	$Q_2 = 0.00137\ A_c^{0\ 790}\ S^{0\ 777}\ I_{24,2}^{8\ 036}$	
10	$Q_{10} = 0\ 00126\ A_c^{0\ 687}\ S^{0\ 683}\ I_{24,2}^{10\ 037}$	
50	$Q_{50} = 0.00240\ A_c^{0\ 632}\ S^{0\ 640}\ I_{24,2}^{10\ 467}$	
100	$Q_{100} = 0.00335\ A_c^{0\ 615}\ S^{0\ 628}\ I_{24,2}^{10\ 491}$	
200	$Q_{200} = 0.00464\ A_c^{0\ 599}\ S^{0\ 618}\ I_{24.2}^{10\ 490}$	
500	$Q_{500} = 0.00755\ A_c^{0\ 581}\ S^{0\ 606}\ I_{24\ 2}^{10\ 393}$	

REGRESSION EQUATION STATISTICS

One way to show the value of updating the regression equations was to use current data in both the new equations and the old ones and then compare the results. This check was performed as previously outlined. The pertinent statistics for each of the regression equations are listed in Table 5.2.

Table 5.2. New regression equation MSE values.

	REGION 1		REGION 2		REGION 3		REGION 4		REGION 5	
Return Period	R^2	MSE of Log	R^2	MSE of Log	R^2	MSE of Log	R^2	MSE of Log	R^2	MSE of Log
2	.518	.995	.641	1.337	.788	.494	.751	.464	.933	.234
10	.406	.700	.686	1.128	.774	.434	.775	.278	.937	.175
50	.265	1.130	.663	1.284	.702	.562	.725	.353	.881	.303
100	.247	1.432	.646	1.394	.661	.655	.695	.419	.849	.376
200	.243	1.790	.628	1.522	.617	.769	.667	.496	.815	.457
500	.250	2.345	.603	1.709	.557	.952	.632	.618	.767	.577

After these regression equations were developed and analyzed, the following observations were made:

1. The value of R^2 was lower for each of the models using the old regression equations. The one exception to this was in Region 1, where the R^2 values were virtually identical.

2. Many of the variables used in the original study are no longer statistically significant in the models. The only models with all of the original study variables statistically significant were the 2- and 10-year models in Region 3, and the 10-, 50-, and 100-year models in Region 4. All other models had at least one variable that was not significant, and many had two variables that were no longer significant.

These results show that the new regression equations are, statistically speaking, an improvement over the Beckman equations.

LIMITATIONS OF REGRESSION EQUATIONS

The regression equations were developed using records for uncontrolled streams. Therefore, these equations are not valid for controlled streams. **Also,** the drainage areas have limits in each region as shown in Table 5.3. Figure 5.12 shows the number of gages in each region falling within different drainage area ranges.

Table 5.3 Limitations of new equations

Region	Minimum Area (mi.^{2})	Maximum Area (mi.^{2})
1	0.4	3300
2	10.0	6430
3	0.4	1590
4	0.4	630
5	1.0	4370

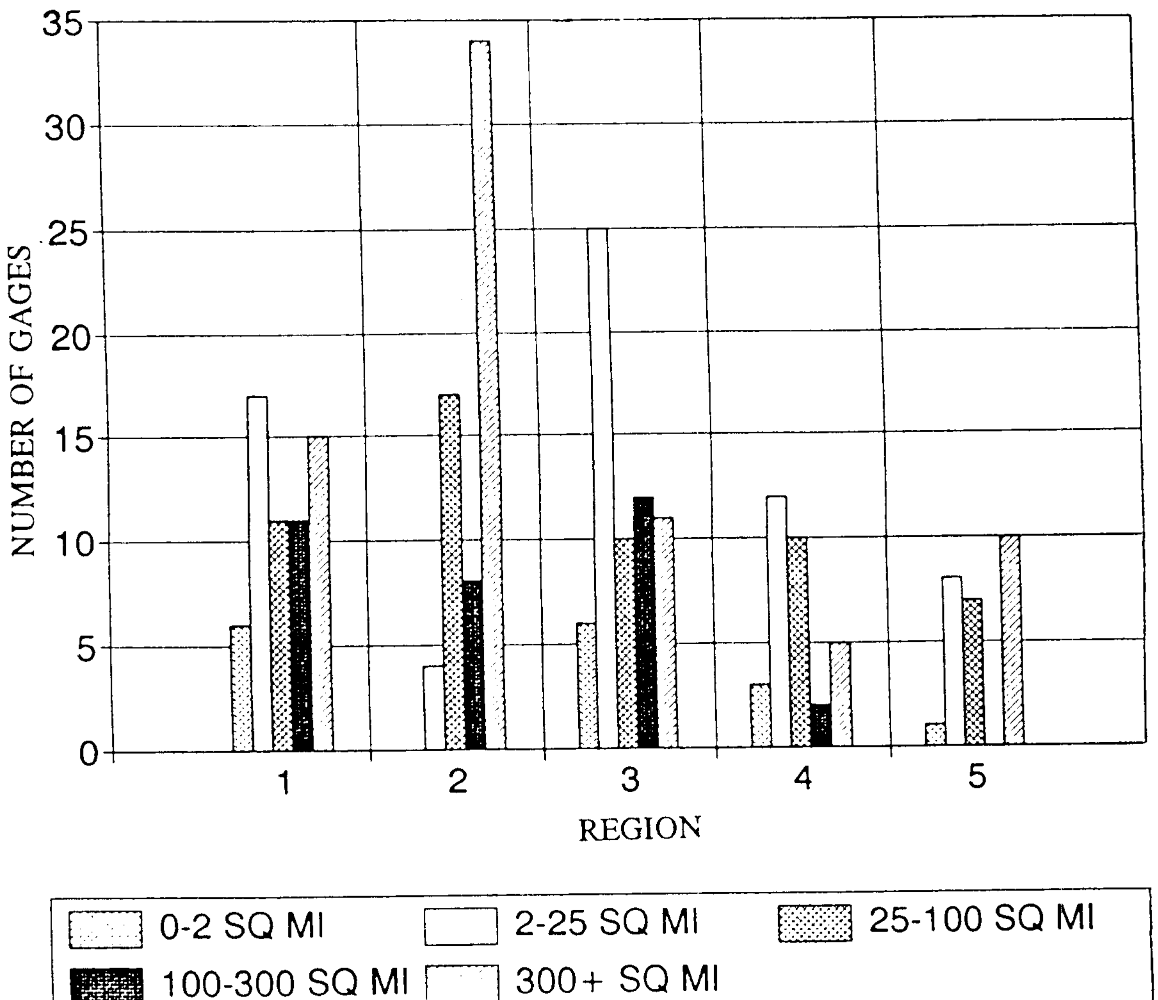


Figure 5.12. Number of gages in each region according to *size* of drainage area.

Chapter 6

COMPARISON WITH NDOR METHODS

The following section compares peak discharges resulting from current NDOR methods to peak discharges obtained by the new regression equations. Three randomly selected watersheds in each of the five hydrologic regions were used in the regression analysis. The appropriate Roadway Design Division or Bridge Division methods for the selected basins were used. The LP3 method was also used, since stream gage data were available for the selected basins. The results of these comparisons are discussed in the next two sections.

Tables 6.1 and 6.2 show the gages used and the results of the peak discharge estimates using the various methods. Table 6.1 is for drainage areas that would probably result in culvert design, and Table ***6.2*** shows drainage areas that would probably result in bridge design. Estimates were made for the 10-, 50-, and 100-year return periods. Any blank value in the table means that the method was not applicable to that basin due to some constraint, such as an oversized drainage area.

ROADWAY DESIGN DIVISION METHODS

Figures 6.1 through 6.3 show bar graphs comparing the estimated peak discharges using the Potter, Rational, LP3, and new regression equation methods. To evaluate the results, it is necessary to have a "true" value for the comparison. Using the LP3 value as the "true" value, several observations can be made.

First, the new regression equations are best, in general, at predicting the LP3 value. This is to be expected since the regression equations are based on the LP3 values. The Rational Method performed well for the longer return periods. For some of the sites, the disparity between LP3 values and estimates from Roadway Design Division methods was 100 perceint or more. However, these differences were not always in the same direction. In other words, the LP3 and regression equations did not predict consistently higher or lower than the other two methods. This finding was true for each of the three return periods examined.

Table 6.1 Estimates of peak discharges for various Roadway Design Division methods.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)		(10)	(11)	(12)	(13)
REF NO	Gage No.	GAGE NAME AND LOCATION	A	Ac	Tp	LP3	Rational	Potter	Cir. 458	WSP 1679/	Beckman	New Reg.
A	6463100	BONE CREEK TRIB NR AINSWORTH	0.39	0.39	10	361	270	250	17		681	312
					50	1810	359	346	26	38	2077	1299
					100	3280	404	392	30	44	3020	2152
B	6463300	SAND DRAW TRIB NR AINSWORTH	1.07	1.07	10	841	360	363	36		523	431
					50	4250	473	502	53	78	1921	1618
					100	7720	534	570	63	90	2525	2584
G	6600800	S. OMAHA CR TRIB NO.2 NEAR WALTHILL	1.65	1.65	10	1470	1030	450	1025		1054	1028
					50	3720	1390	623	1892	2588	2403	2524
					100	5030	1520	706	2328	3208	3199	3615
	6610700	BIG PAPILLION CR NR ORUM	8.52	8.52	10	994		1450	2435		3118	2021
					50	1840		7250	4495	6546	6599	4653
					100	2250		11375	5526	8063	8561	6453
J	6766050	BUFFALO CREEK TRIB NO.1 NR BUFFALO	2.06	2.06	10	148	918	1100	163		864	281
					50	621	1238	1522	300	550	2677	1106
					100	1020	1400	1726	369	656	3678	1804
K	6770700	WOOD RIVER NR LODI	12.9	12.9	10	147		2400	569		158	500
					50	414		3322	1051	1506	436	1282
					100	561		3766	1292	1794	605	1753
L	6782600	S BRANCH MUD CR TRIB NR BROKEN BOW	0.4	0.4	10	248	184	252	52		92	168
					50	543	246	349	97	222	211	397
					100	693	276	395	119	264	273	510
N	6880775	BEAVER CREEK TRIB NR HENDERSON	1.16	1.16	10	49	400	560	109		119	114
					50	88	512	775	201	399	346	303
					100	109	557	879	247	475	489	421
O	6883540	SPRING CREEK TRIB NEAR RUSKIN	2.11	2.11	10	627	650	735	164		294	556
					50	1590	840	1017	303	555	845	1434
					100	2230	920	1153	373	661	1210	1965

(1) REFERENCE LETTER FOR USE WITH BAR GRAPHS
(2) USGS GAGE IDENTIFICATION NUMBER
(3) NAME AND LOCATION OF GAGE IN NEBRASKA
(4) DRAINAGE AREA IN SQUARE MILES
(5) CONTRIBUTING DRAINAGE AREA IN SQUARE MILES
(6) RETURN PERIOD IN YEARS
(7) DISCHARGE ESTIMATE IN CFS USING LOG PEARSON TYPE III METHOD (SECTION 5.1)
(8) DISCHARGE ESTIMATE IN CFS USING RATIONAL METHOD (SECTION 4.1.1)
(9) DISCHARGE ESTIMATE IN CFS USING THE POTTER METHOD (SECTION 4.1.2)
(10) DISCHARGE ESTIMATE IN CFS USING CIRCULAR 458 (SECTION 4.2.1)
(11) DISCHARGE ESTIMATE IN CFS USING WATER SUPPLY PAPER 1679 OR 1680 (SECTION 4.2.2)
(12) DISCHARGE ESTIMATE IN CFS USING BECKMAN REGRESSION EQUATIONS (SECTION 4.1.6)
(13) DISCHARGE ESTIMATE IN CFS USING NEW REGRESSION EQUATIONS (SECTION 5.3)

Table 6.2. Estimates of peak discharges for various bridge design methods.

(1) REF NO	(2) Gage No.	(3) GAGE NAME AND LOCATION	(4) A	(5) Ac	(6) Tp	(7) LP3	(8) Rational	(9) Potter	(10) Cir. 458	(11) WSP 1679	(12) Beckman	(13) New Reg.
C	6464900	KEYA PAHA RIVER NR NAPER	1630	1630	10	5280			15754		6877	4266
					50	9610			29085	7213	17335	8900
					100	11900			35750	8442	24064	11580
D	6777000	MIDDLE LOUP RIVER NR MILBURN	3950	135	10	2130					1131	1157
					50	2680				26650	2890	2478
					100	2910				30750	3880	3315
E	6785000	MIDDLE LOUP AT ST. PAUL	8090	3200	10	19700					23439	29804
					50	37100				44360	55605	69691
					100	47400				51185	76863	95755
F	6791500	CEDAR RIVER NR SPALDING	782	50	10	1550					793	429
					50	2840				8274	1626	718
					100	3580				9550	2151	878
I	6608000	TEKAMAH CREEK AT TEKAMAH	23	23	10	9810		4000	4110		4637	6911
					50	19400		5536	7587	11470	9806	14780
					100	24000		6276	9326	14130	12732	20042
[illegible]	6610700	BIG PAPILLION CR NR ORUM	8.52	8.52	10	994		1450	2435		3118	2021
					50	1840		7250	4495	6545	6599	4653
					100	2250		11375	5526	8063	8561	6453
K	6770700	WOOD RIVER NR LODI	12.9	12.9	10	147		2400	569		158	500
					50	414		3322	1051	1505	436	1282
					100	581		3766	1292	1794	605	1753
M	6880000	LINCOLN CREEK NEAR SEWARD	446	446	10	4350			6474		4865	5426
					50	8130			11952	10629	11289	10974
					100	10100			10629	12668	14950	13969

(1) REFERENCE LETTER FOR USE WITH BAR GRAPHS
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(3) NAME AND LOCATION OF GAGE IN NEBRASKA
(4) DRAINAGE AREA IN SQUARE MILES
(5) CONTRIBUTING DRAINAGE AREA IN SQUARE MILES
(6) RETURN PERIOD IN YEARS
(7) DISCHARGE ESTIMATE IN CFS USING LOG PEARSON TYPE III METHOD (SECTION 5.1)
(8) DISCHARGE ESTIMATE IN CFS USING RATIONAL METHOD (SECTION 4.1.1)
(9) DISCHARGE ESTIMATE IN CFS USING THE POTTER METHOD (SECTION 4.1.2)
(10) DISCHARGE ESTIMATE IN CFS USING CIRCULAR 458 (SECTION 4.2.1)
(11) DISCHARGE ESTIMATE IN CFS USING WATER SUPPLY PAPER 1679 OR 1680 (SECTION 4.2.2)
(12) DISCHARGE ESTIMATE IN CFS USING BECKMAN REGRESSION EQUATIONS (SECTION 4.1.5)
(13) DISCHARGE ESTIMATE IN CFS USING NEW REGRESSION EQUATIONS (SECTION 5.3)

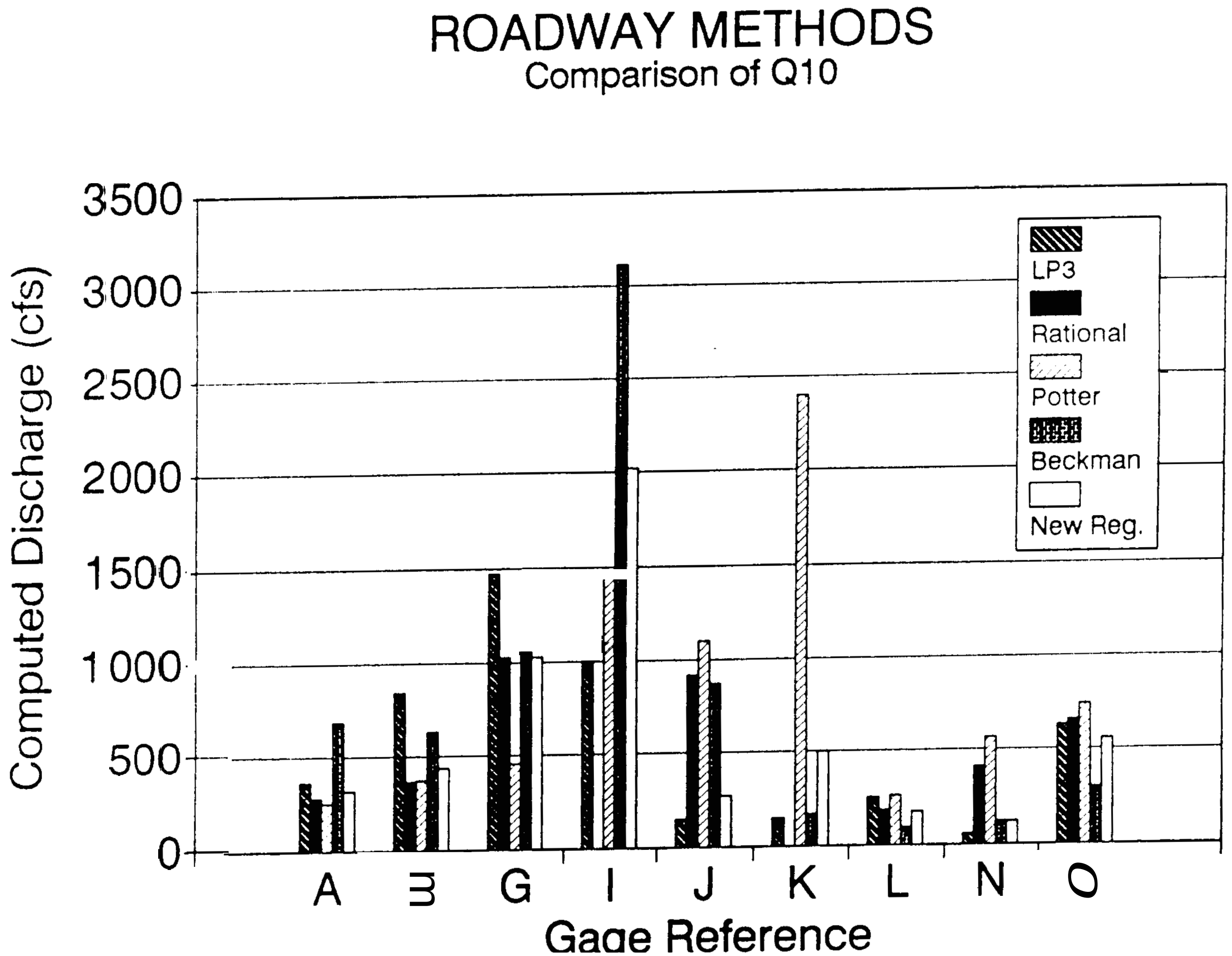

Figure 6.1. Comparison of estimates for 10-year return period for Roadway Design Division methods.

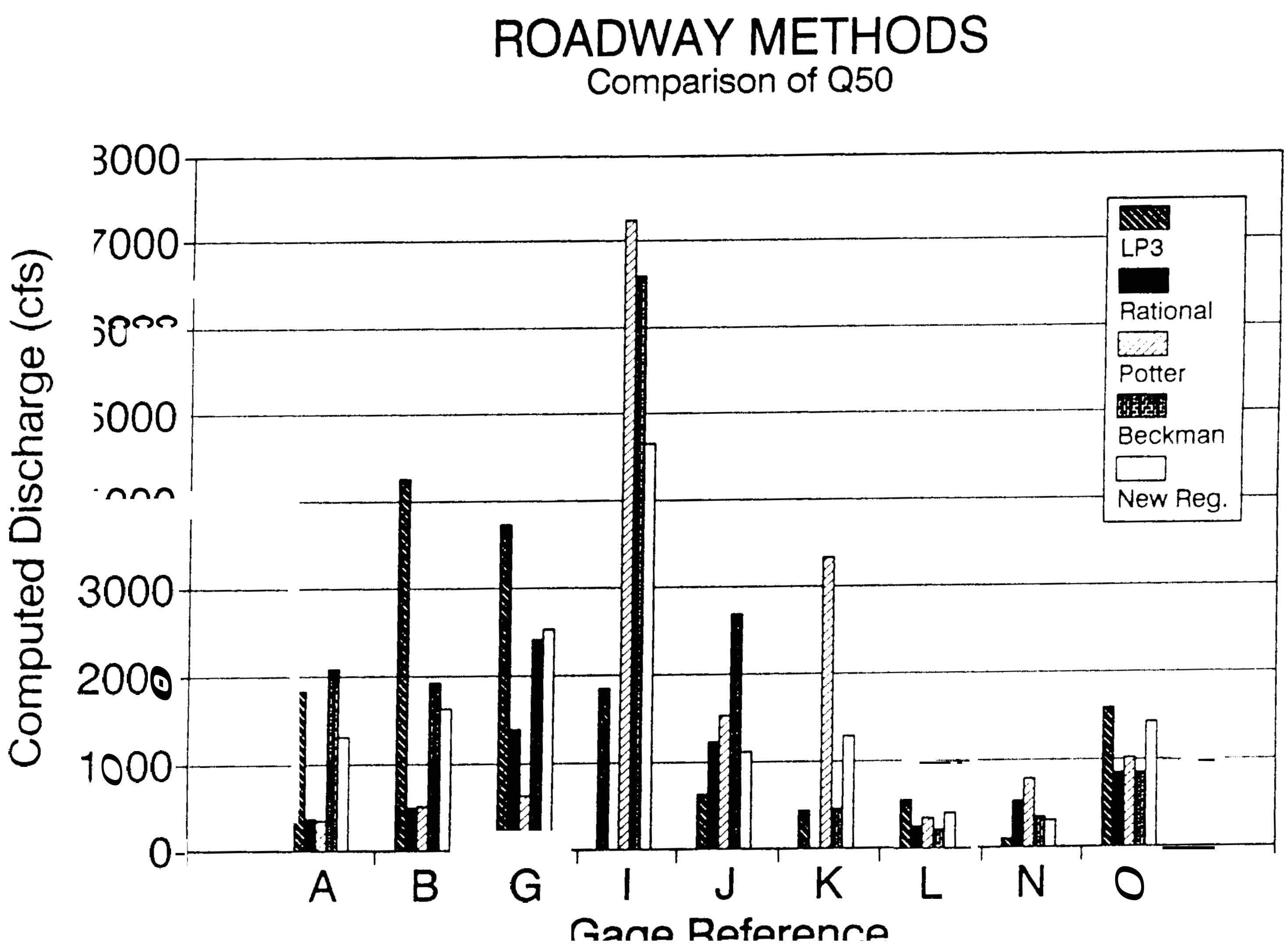

Figure 6.2. Comparison of estimates for 50-year return period for Roadway Design Division methods.

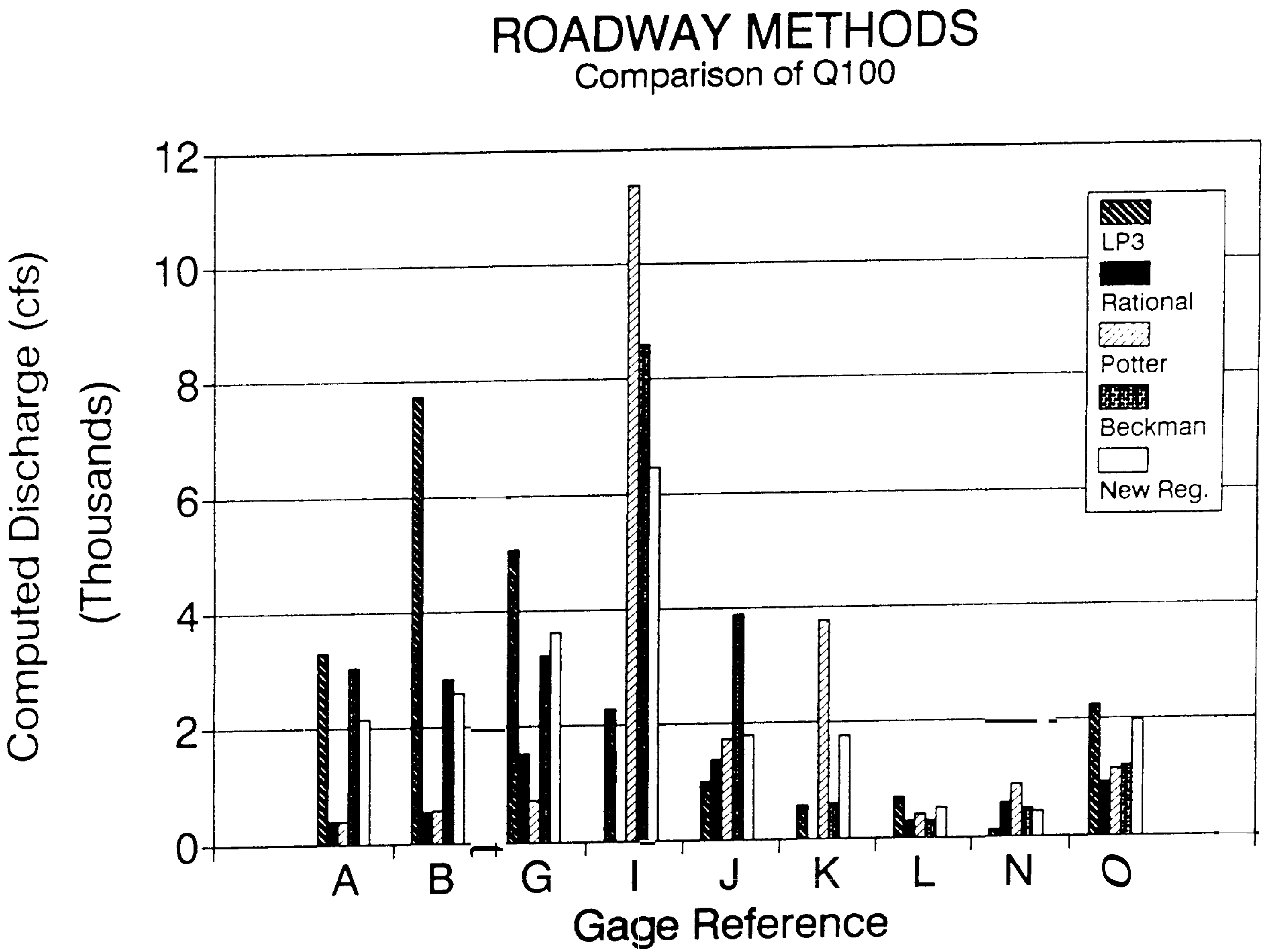

Figure 6.3 Comparison of estimates for 100-year return period for Roadway Design Division methods.

BRIDGE DIVISION METHODS

The comparison of the new regression equations to methods used in the Bridge Division was carried out in the same way as for the Roadway Design Division. Figures 6.4 through 6.6 compare estimates made using LP3, Circular 458, the Beckman regression equations, WSP 1679 or 1680 (depending on location), and the new regression equations. (Refer to Chapter 2 for an overview of these methods.) Figures 6.4 through 6.6 also compare the estimates for 10-, 50-, and 100-year return periods. (The 10-year return period does not show the estimates from WSP 1679 or 1680 because they do not provide for the prediction of a 10-year return period.)

The following general observations can be made about these comparisons. The new regression equations generally are closer to the LP3 estimates than are the results of the other methods, In some instances, however, the Beckman equations are closer. The degree: of variability between all of the methods is generally not as high as it is for the methods used by the Roadway Design Division. There are only a few instances of differences over 100 percent between any two methods at the same site. The greater agreement between methods used by the Bridge Division, as compared to methods used by Roadway Design, may be due to the basis of the former on actual flow data. However, not all Bridge Division methods were based on an LP3 distribution.

No one method consistently predicts higher or lower than the other methods, but the Potter Method tends to predict outside the range of the other methods. This occurs in both the Roadway Design and Bridge calculations. It predicts peaks up to **400** percent different than the LP3 method, and it also performs the worst in each return period. The Circular 458 method also predicts poorly. Figure 6.7 shows the average percentage difference from LP3 for the methods used in both NDOR Divisions.

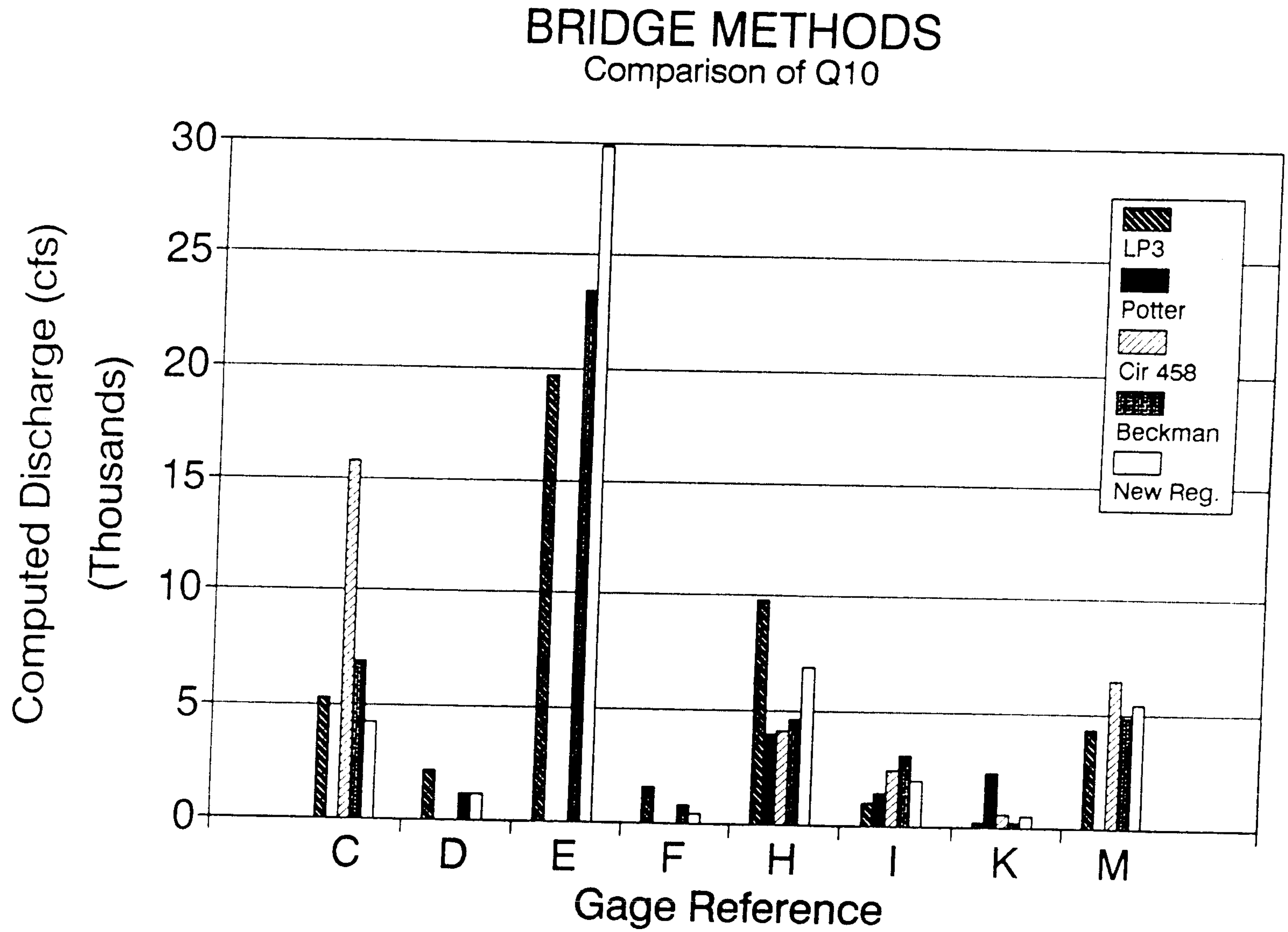

Figure 6.4. Comparison of estimates for 10-year return period for Bridge Division methods.

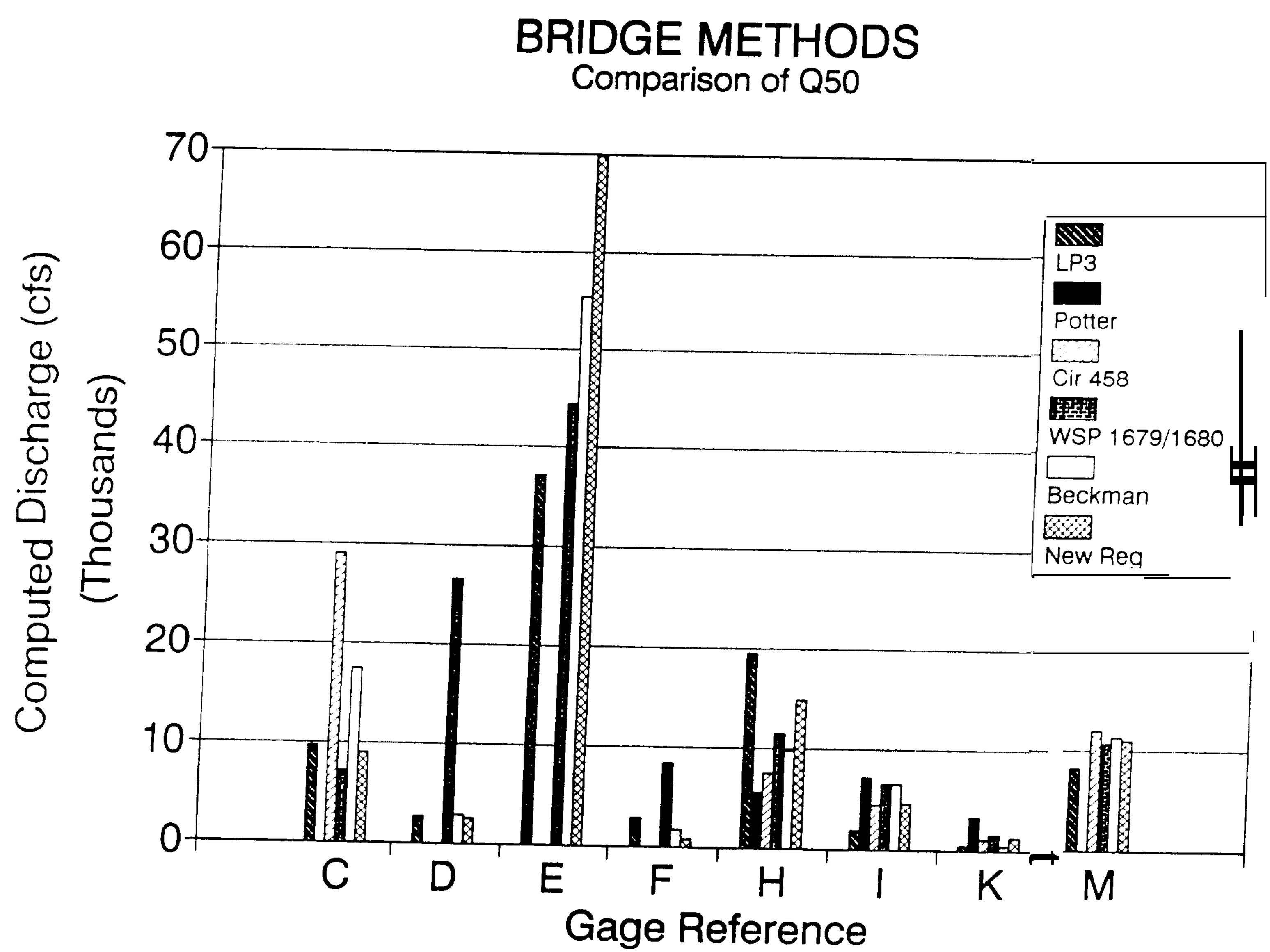

Figure 6.5 **Comparison of estimates for 50-year return period for Bridge Division methods.**

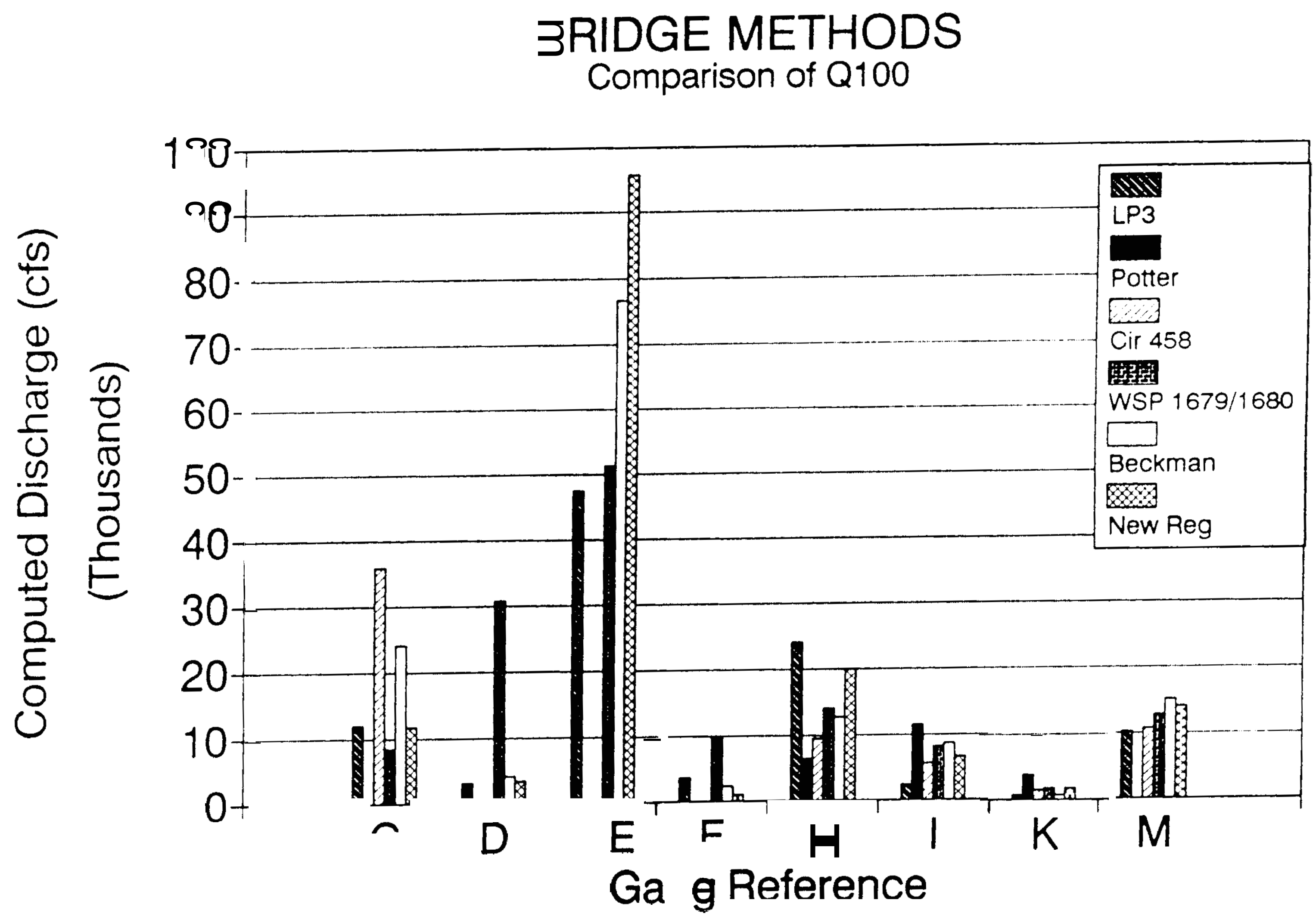

Figure 6.6. Comparison of estimates for 100-year return period for Bridge Division methods.

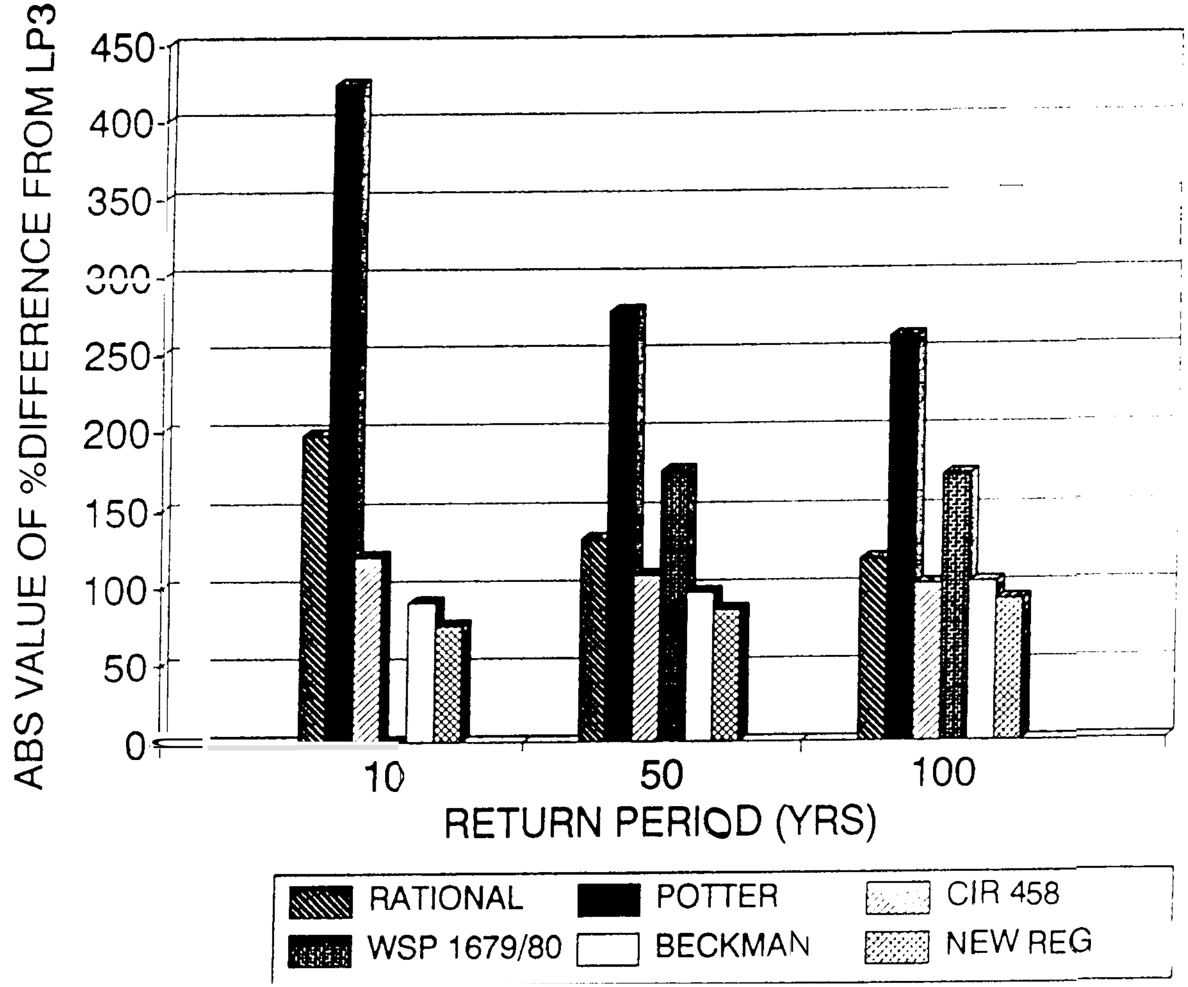

Figure 6.7. **Average percent difference from LP3 for various methods used in the NDOR Bridge and Roadway Design** Divisions.

COMPARISON OF NEW REGRESSION EQUATIONS TO BECKMAN EQUATIONS

This section compares the new regression equations to those developed by Beckman. These comparisons are based on all of the gage sites used to update the regression equations. Among the comparisons performed on the two sets of equations are tests of the sample variances. The sample variance, which is estimated by the mean-square error (MSE), was obtained for each equation by finding the residuals between the observed "true" event (LP3) and the predicted event from each equation. The residuals were then squared and summed. The summation was then divided by the degrees of freedom, which is the number of gages in the region minus the number of estimated parameters. There are four estimated parameters for all of the Beckman equations and for regions 2 through 5 for the new equations. Region 1 (new equations) has three estimated parameters, since only two basin characteristics are used.

McCuen (1993) points out that the regression equations minimize the sum of squares error for the logarithms of the peak flows, but not for the peak flows themselves. This is shown in Table 6.3, where the MSE of logarithms for each of the equations is shown, as well as the root of MSE for the actual, non-transformed peak flows. Although the log MSE values are all smaller for each of the new equations, the root MSE of the non-transformed variables is not. McCuen states that a power equation must be developed to minimize the sum of squares error for the non-transformed variables,. However, power equations are quite complex, and in the literature review, all regional regression equation development procedures utilized the logarithmic transformation (Choquette, 1987; Harris *e? al.*, 1979; Parret, 1981; Schroeder, 1977; and Bridges, 1982).

Table 6.3 shows the median value of the computed differences between the LP3 values, those of the new regression equations, and those of the Beckman equations. Figures 6.9 through 6.13 show box plots of the percentage differences. The top line in the box plot shows the maximum percent difference between the predicted value and the LP3 value in the positive direction. The top of the box shows the point that exceeds 75 percent of the values in that region, and the bottom line of the box shows the point that exceeds 25 percent of the values in the region. Therefore the middle half of the values fall within the box. The line inside or near the box represents the mean value.

Table 6.3 Comparison of mean square error (MSE) for each equation.

REGION	RETURN PERIOD (YRS)	NEW EQ MSE OF LOGS	MEDIAN % DIFF NEW EQ FROM LP3	NEWEQ RMSOF PEAK Q'S (x10000)	BECKMAN MSEOF LOGS	MEDIAN % DIFF BECKMAN FROM LP3	BECKMAN RMSOF PEAK Q'S (x10000)
(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)
1	2	1.00	8.00	0.10	1.24	29.00	0.11
	10	070	3.00	0.28	0.91	23.00	0.32
	50	1.13	-9.00	1.87	1.45	26.00	2.02
	100	1.43	-6.00	6.24	1.83	19.00	6.48
2	2	1.34	15.00	3.49	1.47	18.00	3.73
	10	1.13	15.00	20.10	1.36	13.00	20.50
	50	1.28	30.00	70.90	1.60	1.00	72.00
	100	1.39	4.00	118.30	1.75	-14.00	120.00
3	2	0.44	-16.00	0.20	0.85	-5.00	0.23
	10	0.43	-17.00	0.96	0.65	-1.00	0.71
	50	0.56	-14.00	3.05	0.74	3.00	2.66
	100	0.66	-9.00	5.57	0.83	13.00	5.20
4	2	0.46	9.00	0.02	0.69	4.00	0.03
	10	0.28	3.00	0.08	0.46	5.00	0.15
	50	0.35	-3.00	0.28	0.66	8.00	0.51
	100	0.42	-4.00	0.44	0.78	23.00	0.79
5	2	0.23	-9.00	0.11	0.51	11.00	0.07
	10	0.18	-9.00	0.27	0.41	10.00	0.20
	50	0.30	1.00	0.57	0.53	2.00	0.71
	100	0.38	4.00	0.81	0.80	-18.00	1.10

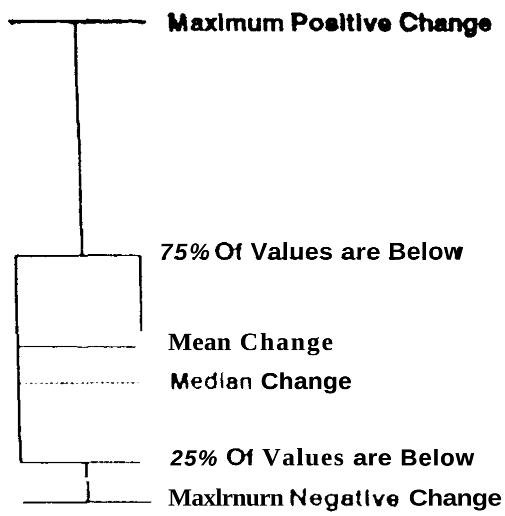

Figure 6.8. Legend for box plots, Figures 6.9 through 6.13.

The bottom line represents the maximum percent difference in the negative direction between the predicted value and the LP3 value. The line on the outside of the box shows the position of the median value. Figure 6.8 displays graphically the legends used in Figures 6.9 through 6.13.

From the MSE and graphical comparisons, one can see that the new equations more accurately predict the **LP3** values. In every region except Region 2, the range of values from the **box** plots is smaller for the new equations than for the old equations. The computed MSE values in Table 6.3 indicate that Region 2 also has less sample variance than with Beckman's equations.

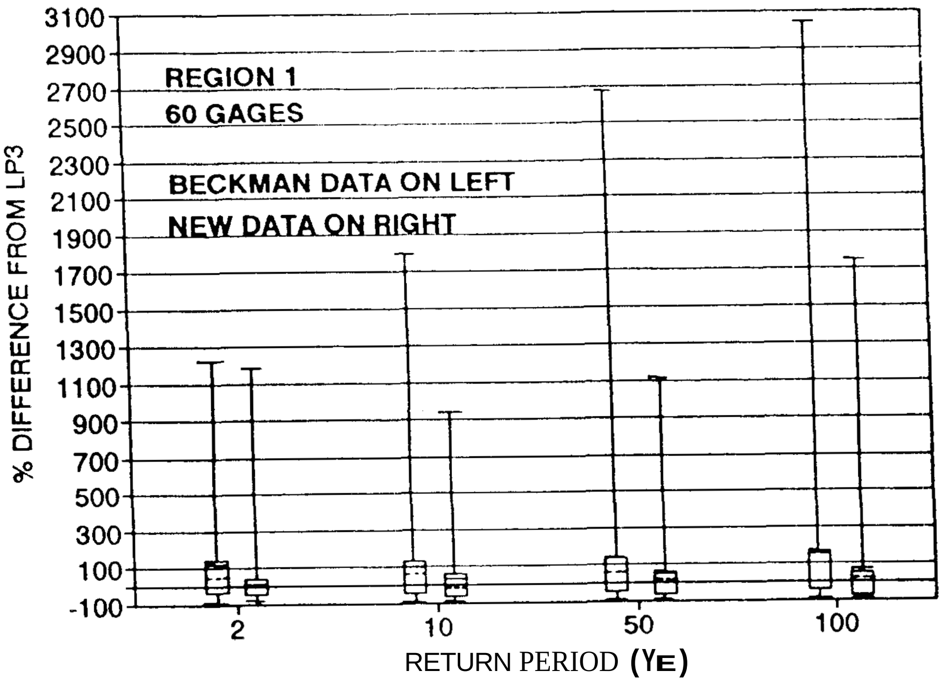

Figure 6.9. Box plot of differences between Beckman's and the new equations for Region 1.

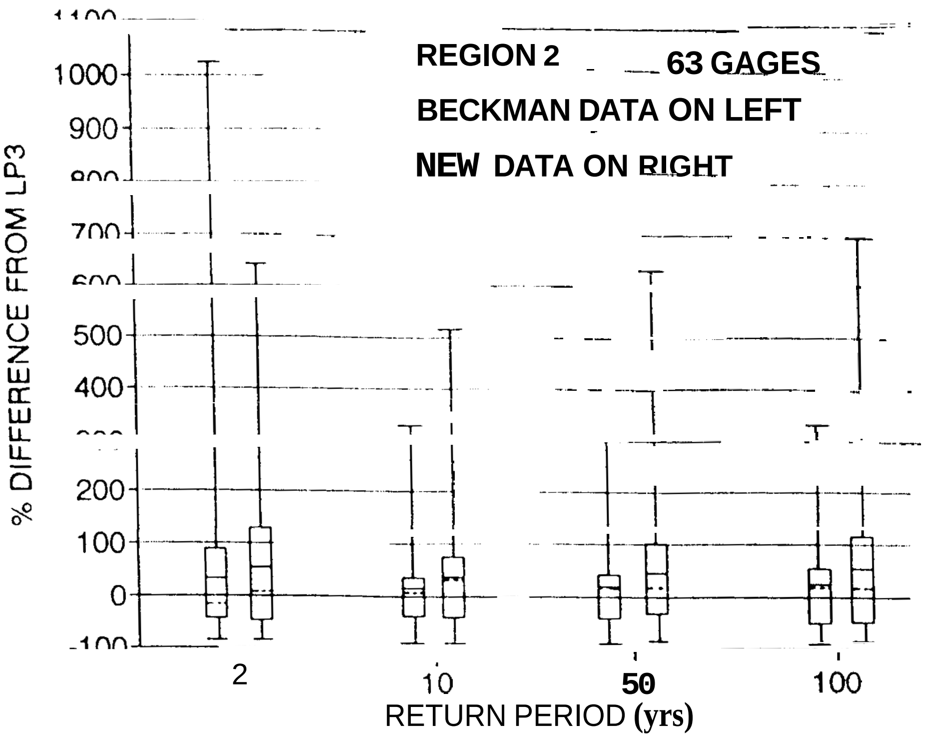

Figure 6.10. Box plot of differences between Beckman's and the new equations for Region 2.

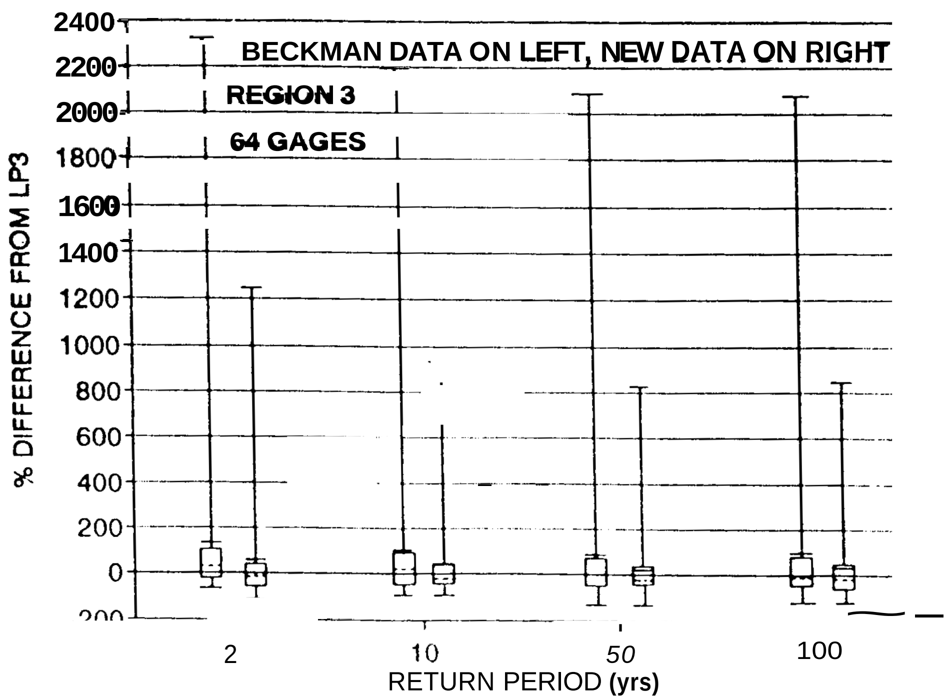

Figure 6.11. Box plot of differences between Beckman's and the new equations for Region 3.

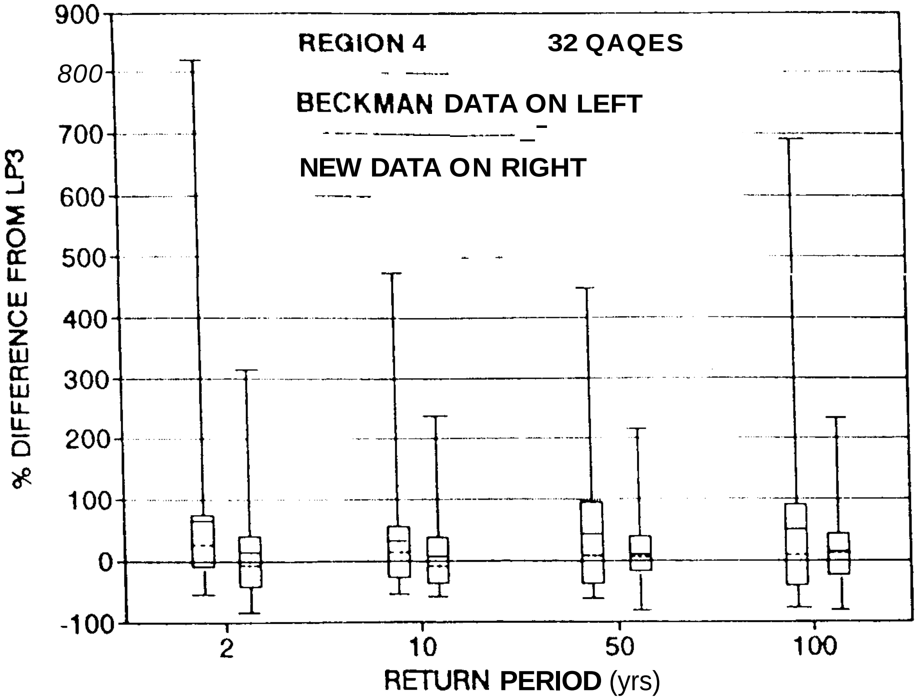

Figure 6.12. Box plot of differences between Beckman's and the new equations for Region 4.

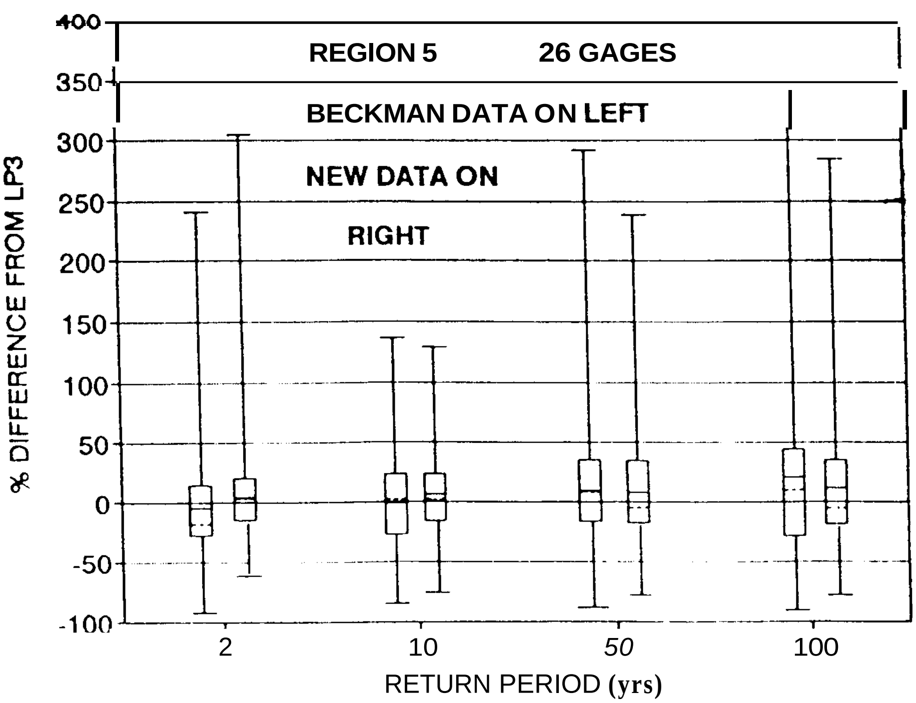

Figure 6.13. Box plot of differences between Beckman's and the new equations for Region 5.

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

The two goals of this study were to make available the most recent methods in hydrologic design, and, if possible, to recommend a uniform design procedure for the two NDOR Divisions. The first section of this chapter presents the conclusions based on this research project. The second section presents recommendations for updating the hydrology section of the Roadway Design Division manual. These recommendations are hased on this research, as well as the previous research performed by Riley (1988) and McCallum (1592). Also, some recommendations for further study are included.

CONCLUSIONS

With regard to the objectives of this research, the following conclusions can be drawn:

1. The new regression equations reflect the most recent methods in design. This is not only because they are newly developed, but also because of the way they were developed. The LP3 stream flow data analysis uses more data than was available when the previous regression equations were developed. Also, improved statistical methods, especially with regard to the generalized skew map, were used to develop the new equations.

2. When stream flow records of sufficient length are available at a site, LP3 analysis should be performed. The generalized skew map presented in this paper shows greater detail, partially because of 25 years of stream flow records as opposed to 10 years of record for the Bulletin 17B map.

3. When the LP3 estimates of peak flow are assumed to be the "true" values, the new regression equations estimate values closest to these. Based on this assumption, the other methods currently used are not as accurate as the new regression equations.

4. The greatest average differences from the LP3 value were from the Potter Method, for all return periods analyzed.

RECOMMENDATIONS

The following recommendations are based upon the results of all three studies that made up this NDOR project.

1. When gage records are available at or near a site, use LP3 in conjunction with the generalized skew map presented in this paper.

2. Both the Roadway Design and Bridge Divisions should use the new regression equations to calculate peak flows.

3. Continue to use the Rational Method for all drainage areas under 0.4 square miles. It should also be used for areas under two square miles as recommended by McCallum (1992) in Region 2, and for areas under one square mile in Region 5. These limits are necessary because no gages from drainage areas smaller than this were used to develop the regression equations. The current NDOR nomograph should be used to compute the time-of-concentration for the Rational Method. A factor of 1.5 should be applied to the time-of-concentration for agricultural watersheds.

4. Replace the current IDF curves with the IDF curves presented by Riley. The Riley curves allow for longer storm durations.

5. Stop using the Potter Method because of its poor estimation of peak flows with respect to LP3, and also because it is not valid for much of the state (Sandhills region).

6. Further research should be conducted to investigate the need to update regional boundaries used for the regression equations. This may result in better regression equations.

7. Further research may be of value to see the effects of using different flow frequency distributions. Specifically, the Wakeby Distribution, which has proven effective in simulation studies, may be a legitimate alternative to the LP3 method.

REFERENCES

American Association of State Highway & Transportation Officials, AASHTO Model Drainage Manual, Task Force on Hydrology and Hydraulics, Washington, DC, 1991.

Beckman, Emil W., and N.E. Hutchison, "Floods in Nebraska on Small Drainage Areas, Magnitude and Frequency," Circular 458, U.S. Geological Survey, Washington, DC, 1962.

Beckman, Emil W., "Magnitude and Frequency of Floods in Nebraska," Water Resources Investigations 76-109, U.S. Geological Survey, Washington, DC, 1976.

Benson, M.A., "Evolution of Methods for Evaluating the Occurrence of Floods," U.S Geological Survey Water Supply Paper 1580-A, Washington, DC, 1962.

Bock, P., G.P. Malhotra, and D.A. Chisolm, "Estimating Peak Runoff Rates from Ungaged Small Rural Watersheds," National Cooperative Highway Research Program Report #136, Highuay Research Board, Washington, DC, 1972.

Bridges, Wayne C., "Techniques for Estimating Magnitudes and Frequencies of Floods on Natural Flow Streams in Florida," U.S. Geological Survey Water Resources Investigations Report 82-4012, Tallahassee, FL, 1982.

Carter, R. W., "Magnitude and Frequency of Floods in Suburban Areas," U.S. Geological Survey Professional Paper 424B, Washington, DC, 1961.

Choquette, Anne F., "Regionalization of Peak Discharges for Streams in Kentucky," U.S. Geological Survey Water Resources Investigations Report 87-4209, Louisville, KY, 1988.

Creighton, David E. Jr., "Separation of Skewness: Reality or Regional Artifact?" Journal of Hydraulic Engineering, ASCE, 119(4), pp. 535-538, 1993.

Dalrymple, Tate, "Flood-Frequency Analysis," Manual of Hydrology, Part 3, U.S. Geological Survey Water-Supply Paper 1543-A, Washington, DC, 1960.

Gupta, Ram S., Hydrology and Hydraulic Systems, Prentice Hall, Englewood Cliffs, NJ, 1989.

Gumbel, E.J., Theory of Extremes, Columbia University Press, New York, NY, 1958.

Hardison, Clayton H., "Prediction Error of Regression Estimates of Streamflow Characteristics at Ungaged Sites," U.S. Geological Survey Professional Paper 750-C, pp. C228-C236, 1971.

Hardison, Clayton H., and Marshall E. Jennings, "Bias in Computed Flood Risk," Journal of the Hydraulics Division, ASCE, 98(HY3), pp. 415-427, 1972.

Harris, D.D., Hubbard, Larry L., and Hubbard, Lawrence E., "Magnitude and Frequency of Floods in Western Oregon," U.S. Geological Survey Open File Report 79-553, Portland, OR, 1979.

Hosking, J. R. M. and J. R. Wallis, "Some Statistics Useful in Regional Frequency Analysis," Water Resources Research, 29(2), pp. 271-281, 1993.

Hotchkiss, Rollin H., and Kent Cordes, "Additional Years of Record and Peak Discharge," in Proceedings ASCE Hydraulics Division, Engineering Hydrology Symposium, July 25-30, 1993, pp.533-538.

Houghton, John C., "Birth of a Parent: The Wakeby Distribution for Modeling Flood Flows," Water Resources Research, 14(6), pp. 1105-1109, 1978.

Hromadka, T.V. II, and Robert Whitley, "Testing for Nonzero Skew in Maximum Runoff Data," Water Resources Research, 29(2), pp.531-534, 1993.

Kerridge, D., "Errors of Prediciton in Multiple Regression with Stochastic Regressor Variables," Technometrics, 9(2), pp. 309-311, 1967.

Matalas, N.C., and E.J. Gilroy, "Some Comments on Regionalization in Hydrologic Studies," Water Resources Research, 4(6), pp. 1361-1369, 1968.

Matthai, H.F., "Magnitude and Frequency of Floods in the United States," Water Supply Paper 1680, U.S. Geological Survey, Washington, DC, 1968.

McCallum, Brian E., "Hydrologic and Hydraulic Design of Culverts," University of Nebraska-Lincoln Graduate Thesis, Lincoln, NE, 1992.

McCuen, Richard H., Walter J. Rawls, Gary T. Fisher, and Robert L. Powell, "Flood Flow Frequency For Ungaged Watersheds: A Literature Evaluation," Agricultural Research Service, U.S. Department of Agriculture, ARS-NE-86, November, 1977.

McCuen, Richard H., Microcomputer Applications in Statistical Hydrology, Prentice Hall, Englewood Cliffs, N.J., 1993.

Nebraska Department of Roads, Roadway Design Manual, Lincoln, NE, 1984.

Neter, John, William Wasserman, and Michael H. Kutner, Applied Linear Statistical Models, Irwin, Inc., Boston, MA, 1990.

Parret, Charles, and Omang, R.J., "Revised Techniques for Estimating Magnitude and Frequency of Floods in Montana," U.S. Geological Survey Open File Report 81-971, Helena, MT, 1981.

Patterson, James L. "Magnitude and Frequency of Floods in the United States," Water Supply Paper 1679, U.S. Geological Survey, Washington, DC, 1966.

Ponce, Victor M. Engineering Hydrology: Principles and Practices, Prentice Hall, Englewood Cliffs, NJ, 1993.

Riggs, H.C., "Some Statistical Tools in Hydrology," Techniques of Water-Resources Investigations, Book 4, Chap A1, U.S. Geological Survey, Washington, DC, 1967.

Riggs, H.C., "Frequency Curves," Techniques of Water-Resources Investigations, Book 4, Chapter A2, U.S. Geological Survey, Washington, DC, 1968.

Riggs, H.C., "Regional Analysis of Streamflow Characteristics," Techniques of Water Resources Investigations, Book 4, Chap B3, U.S. Geological Survey, Washington, DC. 1973.

Riley, T.E., "A Hydrologic Evaluation of Twenty-Four Small Watersheds in Nebraska," University of Nebraska-Lincoln Graduate Thesis, Lincoln, NE, 1988.

SAS Institute Inc. SAS/STAT User's Guide, Release 6.03 Edition. Cary, NC: SAS Institute Inc., 1988.

Schroeder, E.E., and Massey, B.C., "Technique for Estimating the Magnitude and Frequency of **Floods** in Texas," U.S. Geological Survey Water Resources Investigations Report 77-110, Austin, TX, 1977.

Singh, Vijay P., Elementary Hydrology, Prentice Hall, Englewood Cliffs, NJ, 1992.

Tasker, Gary D., "Flood Frequency Analysis with a Generalized Skew Coefficient," Water Resources Research, 14(2), pp. 373-376, 1978.

U.S. Soil Conservation Service, Urban Hydrology for Small Watersheds, Technical Release 55, Second Edition, U.S. Department of Agriculture, Washington, DC, 1986.

U.S. Water Resources Council, Guidelines for Determining Flood Flow Frequency, Bulletin #15 of the Hydrology Committee, U.S. Water Resources Council, Washington, DC, 1967.

U.S. Water Resources Council, Guidelines for Determining Flood Flow Freauencv, Bulletin #17B of the Hydrology Committee, U.S. Water Resources Council, Washington, DC, 1981.

U.S. Weather Bureau, Climates of The States, Washington, DC, 1959.

U.S. Weather Bureau, Evaporation Maps for the United States, Technical Paper No. 37, Washington, DC, 1959.

U.S. Weather Bureau, Rainfall Frequency Atlas of The United States, Technical Paper 40, Washington, DC, 1961.

U.S. Weather Bureau, Frequency of Maximum Water Equivalent of March Snow Cover in North Central United States, Technical Paper No.50, Washington, DC, 1964.

Victorov, Peter, "Effect of Period of Record on Flood Prediction," Journal of the Hydraulics Division, ASCE, 97(HY11), pp. 1853-1856, 1971.

Wallis, James R., and Eric F. Wood, "Relative Accuracy of Log Pearson III Procedures," Journal of Hydraulic Engineering, ASCE, 111(7), pp. 1043-1056, 1985.

APPENDIX I

Gages with Log Pearson Computed Peak Flows

HYD REG	GAGE ID	YEARS REC	LOG PEARSON PEAKS (CFS)					
			Q2	Q10	Q50	Q100	Q200	Q500
1	6396490	10	54	209	494	077	000	1300
1	6443200	18	20	431	2300	4370	7730	15600
1	6443300	26	I8	373	2560	5100	9080	22400
1	6443700	24	on	1400	8000	17200	31900	68400
1	6444000	59	312	1430	4020	5920	8510	13400
1	6445000	13	804	2440	5360	7230	9590	13700
1	6445500	18	1180	3550	7250	9420	12000	16200
1	6445560	20	35	1140	10900	24800	53500	138000
1	64.53400	14	738	2310	4370	5430	6500	8290
1	6453500	43	1280	4300	8510	10700	13000	16500
1	6453600	34	1490	5460	11900	15000	20100	27200
1	6456300	18	13	1720	39000	122000	351000	1290000
1	6456400	29	21	25x0	81800	308000	1090000	5410000
1	6457700	18	70	344	1030	1550	2280	3690
1	6463080	12	300	529	742	835	930	1060
1	6463100	13	31	361	1810	3280	5740	11500
1	6463200	11	66	441	1690	2810	4570	8410
1	6463300	19	71	841	4250	7720	13500	26900
1	6464900	34	1940	5280	9610	11900	14400	18100
1	6465300	21	12	130	648	1190	2100	4290
1	6465310	13	591	2290	5550	7680	10400	15100
1	6465400	11	18	115	326	467	646	955
1	6465440	11	564	1960	4400	5920	7810	11000
1	6465680	12	130	295	473	557	646	771
1	6677500	61	7310	20800	41000	52600	66200	88000
1	6685000	61	285	8040	122000	366000	1060000	4210000
1	6762500	60	202	2230	11400	21100	37600	77400
1	6763500	30	53	413	2280	4550	8950	21500
1	6767100	20	199	871	1990	2640	3390	4580
1	6767200	27	22	127	355	509	707	1050
1	6767300	22	112	2340	14300	26900	48000	96600
1	6767400	21	64	990	4360	7150	11100	18600
1	6767410	29	162	1580	5680	8770	13000	20600
1	6767500	32	374	1540	3030	3740	4470	5460
1	6829700	13	275	1130	2870	4040	5560	8250
1	6836500	46	584	2400	6200	8680	11900	17400
1	0839200	27	193	1210	3950	6080	0080	14900
1	6839400	19	474	3610	13300	21300	33000	56700
1	6830600	19	080	4410	14600	22500	33700	55600
1	6839850	10	115	1630	0030	11300	17300	28900
1	6830000	28	245	1400	3510	4760	0230	8530
1	0830050	28	334	018	1750	2210	2740	3580
1	6840000	32	405	1820	3880	4950	6120	7800
1	0840500	20	839	5200	17800	28000	42800	72600

HYD REG	GAGE ID	YEARS REC	LOG PEARSON PEAKS (CFS)					
			Q2	Q10	Q50	Q100	Q200	Q500
1	6840500	20	839	5260	17800	28000	42800	72600
1	6841500	26	495	2850	7460	10300	13800	19300
1	6844000	37	1060	4410	10600	14500	19300	27400
1	6845200	36	1390	4350	8640	1 1000	13700	17900
1	6847000	55	300	2500	8120	12100	17300	26400
1	6847500	47	746	4700	14900	22600	33100	53000
1	6849600	13	592	1620	2750	3270	3810	4560
1	6850000	34	561	1790	3050	3590	4110	4780
1	6850200	26	223	688	1210	1440	1690	2010
1	6851000	36	474	1810	3120	3630	4100	4660
1	6851100	18	161	746	1560	1960	2380	2970
1	6851200	18	244	1760	4660	6340	8280	11200
1	6851300	26	214	692	1270	1560	1860	2270
1	6851400	26	389	1310	2520	3130	3800	4780
1	6851500	36	1790	5150	9240	11300	13400	16500
1	6852000	38	982	3340	7550	10200	13600	19300
1	6853100	40	195	648	1270	1600	1970	2510
2	6454000	36	52	402	1710	2960	4990	9620
2	6454100	34	53	112	188	230	278	352
2	6454500	45	170	665	1950	3000	4550	7750
2	6457200	26	15	181	753	1230	1920	3280
2	6457500	50	834	3530	9550	13000	10800	30900
2	6459200	20	430	633	803	1030	1180	1420
2	6459500	45	401	804	1520	1980	2580	3650
2	6460900	17	59	134	235	289	352	451
2	6461000	44	209	517	1060	1400	1850	2630
2	6461500	46	2070	5930	9840	12000	13400	18200
2	6462000	32	2550	5080	8520	10400	12700	16200
2	6462500	43	433	1060	2020	2590	3270	4390
2	6463500	42	991	3300	7700	10600	14400	21100
2	6465000	65	206000	1240000	4960000	8590000	14600000	28700000
2	6465500	36	202000	1030000	2010000	3600000	4810000	6810000
2	6478520	14	367	2780	8260	11800	16300	23700
2	6678000	59	1030	5200	9200	11400	13600	17000
2	6679000	31	3320	10000	22100	30000	40000	57700
2	6687000	61	2110	5160	11100	15200	20700	30800
2	6692000	60	357	677	1150	1430	1770	2330
2	6775500	46	721	1020	1390	1580	1780	2090
2	6775900	25	312	565	045	1170	1440	1890
2	6776500	47	530	796	1110	1280	1460	1740
2	6777000	10	1520	2130	2680	2910	3150	3480
2	6777500	20	1850	2600	3350	3700	4070	4590
2	6779000	54	2940	6230	11500	14800	18800	25700
2	6780000	17	3020	6830	12300	15400	19100	25100
2	6782500	26	3740	13000	29600	40300	53700	76800
2	6782700	27	28	354	1880	3490	6220	12700

HYD REG	GAGE ID	YEARS REC	LOG PEARSON PEAKS (CFS)					
			Q2	Q10	Q50	Q100	Q200	Q500
2	6782800	17	55	Y32	4160	6830	10600	17600
2	6784000	48	30.50	11100	26100	35000	48400	70000
2	6785000	69	8260	19700	37100	47400	59900	80700
2	6786000	55	1430	2250	3110	3530	3980	4620
2	6787.500	51	603	900	1380	1500	1810	21.50
2	6788500	42	2750	5300	8560	10300	12300	1.5400
2	6780000	33	5400	15300	31000	41000	54100	75200
2	6790500	07	5780	16100	34300	45900	60700	86400
2	6701500	43	651	1550	2840	3580	4450	5840
2	6792000	52	2860	9070	19500	25900	33900	47100
2	6703000	52	14300	38600	77500	101000	130000	177000
2	6704000	51	2280	7260	15500	20500	26600	36800
2	6794500	66	17100	43800	78900	97500	119000	151000
2	6796978	11	298	991	1960	2470	3050	3920
2	6797500	45	1290	5000	11300	15000	19500	26700
2	6798000	34	477	1970	4020	6860	9350	13700
2	6798300	17	330	919	1690	2110	2580	3290
2	6798500	59	1880	6900	16300	22100	29400	41800
2:	6799000	59	4280	12300	22700	28100	34200	43200
2	6800500	74	11700	32200	61300	77400	96100	125000
2	6821500	60	1440	10300	35300	54700	82100	135000
2	6823000	61	231	780	1880	2630	3620	5430
2	6823500	51	26	64	121	155	197	266
2	6824000	51	40	110	239	326	441	647
2	6831000	22	378	1390	3350	4650	6340	9320
2	6831500	51	167	691	2000	3030	4500	7460
2	6835000	42	242	1000	2710	3940	5620	8770
2	6835100	13	358	3300	16300	30200	54100	113000
2	6835500	21	1180	4110	10800	15800	22800	36500
2	6836000	42	385	1520	3530	4760	6270	8780
2	6838000	22	1880	8120	23600	3.5500	52500	86100
2	6839000	27	209	856	2380	3520	5120	8220
2	6839500	27	905	11000	49400	83800	136000	244000
2	6841000	43	1470	5970	14600	20100	27100	39200
3	646.5850	11	14	88	250	379	538	810
3	6466500	41	3240	17.500	46700	0.5000	80400	130000
3	647851X	13	2910	10900	25800	35400	47700	69000
3	6600600	18	404	1070	1960	2430	2960	3780
3	6600700	18	1260	6770	18600	26500	36600	54300
3	6600800	29	252	1470	3720	5030	6580	8970
3	6600000	2x	1880	7200	16400	21900	28600	39700
3	6601000	47	3350	31300	192000	398000	809000	2020000
3	6607700	18	618	3230	10200	15800	23800	39800
3	6607800	20	566	3400	12000	19100	29400	50700

HYD REG	GAGE ID	YEARS REC	LOG PEARSON PEAKS (CFS)					
			Q2	Q10	Q50	Q100	Q200	Q500
3	6607900	18	1220	2830	4860	5920	7110	8910
3	6608000	40	2390	9810	10400	24000	28800	35400
3	6608600	16	279	1980	5360	7370	9740	13400
3	6608700	28	243	1000	2220	2910	3700	4940
3	6608800	25	936	3270	6290	7790	9410	11700
3	6608900	20	740	3520	9270	13100	18000	26600
3	6609000	25	1340	7000	24200	39300	62500	113000
3	6610700	11	308	994	1840	2250	2680	3290
3	6795000	19	1870	9810	24600	33700	44500	61900
3	6795500	43	1610	4190	0950	8220	9520	11300
3	6799080	16	139	477	970	1240	1550	2010
3	6799100	31	1340	5510	13700	19200	26200	38500
3	6799190	12	547	1610	2750	3260	3770	4460
3	6799230	13	2180	8780	19300	25300	32200	42900
3	6799385	13	6090	10000	30500	51700	66300	90200
3	6790423	13	304	2060	7300	11600	18100	31100
3	6799500	52	5980	15300	25100	29500	34000	40200
3	6800000	41	2800	9020	18800	24500	31300	42300
3	6800350	11	67	318	683	870	1070	1360
3	6803000	42	2050	10100	26700	37700	51700	75800
3	6803500	42	8710	23900	37400	42800	47800	54000
3	6803510	23	1570	6240	14800	20200	26900	38100
3	6803520	23	1540	7040	15700	20400	25800	33800
3	6803530	22	2720	8100	16100	20500	25600	33700
3	6803540	17	819	4080	8230	10100	12000	14400
3	6803555	40	13700	44100	75400	88700	102000	118000
3	6803570	29	210	823	1510	1810	2100	2470
3	6803600	28	1070	10800	35200	51500	71700	105000
3	6803700	18	1400	5340	11900	15800	20500	28000
3	6803900	28	1830	12400	39400	50100	85800	135000
3	6804000	42	4460	21600	60300	87800	125000	192000
3	6804100	20	487	2220	4470	5520	6600	8040
3	6804200	29	623	4120	11500	16300	22200	31800
3	6804300	28	81	694	2720	4450	7040	12400
3	6804400	28	108	1050	3600	5630	8220	12900
3	6804500	29	719	6160	19900	29600	42000	63600
3	6805000	21	18100	41300	61300	69300	76900	86500
3	6806400	19	1500	9020	19900	25200	30700	38200
3	6806420	19	1090	4530	8030	9410	10700	12200
3	6806440	29	1320	5040	9310	11200	13100	15600
3	6806460	30	2580	13200	27500	34100	40900	49800
3	6806470	20	285	970	1710	2010	2320	2700
3	6806500	42	5460	22300	45100	56400	68500	85500
3	6810060	11	540	1700	3390	4220	5150	6500

HYD REG	GAGE ID	YEARS REC	LOG PEARSON PEAKS (CFS)					
			Q2	Q10	Q50	Q100	Q200	Q500
3	6810100	20	593	3140	6510	8060	0020	11700
3	0X10200	18	3140	10500	22200	29100	37300	5(1700
3	6810300	18	1830	8020	18100	23000	30500	40800
3	0X10400	20	190	617	1000	1260	1450	1700
3	6810500	20	7690	37000	03300	128000	169000	238000
3	6811500	42	15500	55400	101000	122000	143000	170000
3	0X14500	39	15800	43900	67700	76600	84700	94400
3	6815000	49	23700	49700	67200	73100	78300	84100
3	0815500	21	0240	23600	39800	47500	55600	07000
3	0815510	11	132	1640	5690	8440	11900	17500
4	6708050	14	12	14X	621	1020	1600	2740
4	6708100	2x	11	158	501	836	1180	1740
4	6768200	17	93	496	1400	2030	2860	4340
4	6768400	28	27	240	829	1240	1780	2720
4	6768500	23	197	1350	4790	7630	11800	20200
4	6769000	10	140	332	501	569	634	710
4	6769100	28	50	160	265	308	348	399
4	0769200	28	38	382	1260	1850	2600	3860
4	6770700	27	20	147	414	581	784	1110
4	67708110	28	119	887	2430	3340	4420	6080
4	6770900	28	110	834	2310	3200	4260	5890
4	6770910	27	186	1070	2780	3840	5110	7150
4	6771000	35	502	3250	11200	17700	27300	46500
4	6771500	28	560	2020	3910	4840	5830	7220
4	6772000	38	360	959	1600	1890	2190	2590
4	6777700	29	45	873	5030	9290	16200	31800
4	6778000	20	1790	2630	3540	3970	4440	5120
4	6782600	28	47	248	543	693	854	1080
4	6782900	29	35	598	3090	5460	9150	17000
4	6783500	45	795	3150	8120	11600	16200	24700
4	6784300	14	318	1020	1950	2430	2960	3740
4	6784700	27	251	2210	5880	7860	10000	13100
4	6788988	12	208	1910	7450	1210	18900	32500
4	0789100	17	175	1870	5750	8130	10900	15200
4	6780210	20	117	920	2060	4400	6270	9550
4	6789300	17	461	1970	3990	4980	6030	7400
4	6789400	28	218	1560	4300	6100	8200	11000
4	6790000	27	72	534	1450	1980	2010	3570
4	6790700	2x	400	4800	15000	20800	27500	37200
4	6700800	17	1030	4110	9210	12200	15700	21300
4	6790900	16	205	1300	4160	6340	9360	15100
4	6791100	31	872	4810	13100	18500	25400	37200
5	6704710	12	570	3910	13700	21800	33400	56900
5	6880000	38	1440	4350	8130	10100	12200	15300

HYD REG	GAGE ID	YEARS REC	LOG PEARSON PEAKS (CFS)					
			Q2	Q10	Q50	Q100	Q200	Q500
5	6880500	38	3120	9620	17300	20900	24700	29900
5	6880508	12	760	4160	12500	18700	27200	43300
5	6880590	11	270	933	1750	2140	2550	3120
5	6880710	19	37	531	2140	3370	5040	8020
5	6880720	26	27.5	1280	2880	3760	4760	6270
5	6880730	26	174	526	1010	1270	1560	2010
5	6880740	19	535	2080	4270	5400	6650	8480
5	6880775	11	18	40	88	109	133	167
5	6880800	35	3270	10800	24000	32300	42600	60300
5	6881000	47	6760	19600	35000	42500	50600	61900
5	6881200	32	2190	7910	18800	26000	35300	51500
5	6881450	29	1200	5380	9710	11400	13000	14800
5	6881500	88	9040	28700	52600	64100	76300	93400
5	6882000	71	14000	32500	50200	57800	65500	75600
5	6883000	39	4080	12000	23000	29000	35800	46200
5	6883540	12	150	627	1590	2230	3060	4530
5	6883570	32	5940	14900	25600	30900	36700	45200
5	6883600	18	85	653	1910	2710	3690	5300
5	6883700	28	248	1280	3180	4340	5730	7960
5	6883800	19	340	1700	3950	5190	6600	8730
5	6883900	19	785	2130	3810	4670	5600	6070
5	6883955	11	340	913	1750	2230	2790	3690
5	6884000	71	8200	23100	**42400**	52400	63600	80200
5	6884005	11	200	1330	3320	4420	5640	7430

APPENDIX II

Gages and Basin Characteristics

HYD REG	GAGE ID	A (mi ^ 2)	Ac (mi ^ 2)	S (ft/mi)	L (mi)	P (in)	I2,24 (in)	I50,24 (in)	SN10 (in)	T1 AH	T3 FAHR	T4 FAHR	EVAP (in)
1	6396490	24.5	24.5	5.95	6.22	16.2	1.9	3.9	0.7	0.8	47	89	44.9
1	6443200	7.97	7.97	94.7	5.78	17	1.92	4	0.7	10	46	89	44.8
1	6443300	10.9	10.9	83.3	8	17	1.92	4	0.7	10	47	89	44.8
1	6443700	52.6	52.6	56.2	15.88	17.5	1.93	4	0.9	10	46	89	44.8
1	6444000	313	313	34.8	34.2	17.3	1.93	4	0.8	10	46	89	45
1	6445000	676	676	24.5	60.4	17	2.01	4.1	0.8	10	46	90	44.5
1	6445500	750	750	22.8	65.7	17	2.01	4.1	0.8	10	46	90	44.5
1	6445560	15.4	15.4	78.5	5.38	17.3	2.02	4.2	0.95	10	46	90	44
1	6453400	373	373	5.11	81	21.2	2.4	4.9	2.1	9	42	91	38.5
1	6453500	505	505	5.89	104	21.3	2.42	5	2.1	9	42	91	38.8
1	6453600	812	812	6.96	152	21.7	2.44	5.1	2.2	9	42	92	38.8
1	6456300	23.5	23.5	29.6	15.74	17.3	2.02	4.2	0.8	10	46	89	44.5
1	6456400	82.2	82.2	24.3	21.9	17.2	2.02	4.2	0.8	10	46	89	44.5
1	6457700	61.1	61.1	9.1	190	17	2.08	4.3	1.05	7	41	91	42
1	6463080	246	246	12.3	26.6	20.9	2.4	5	1.7	0.5	43	90	40.9
1	6463100	0.39	0.39	31.9	0.46	22.1	2.33	4.9	1.7	10	44	90	40.5
1	6463200	2.18	2.18	17.3	3.11	22.1	2.34	4.9	1.7	10	42	90	40.5
1	6463300	1.07	1.07	18.8	1.77	22	2.33	4.9	1.7	10	44	90	40.5
1	6464900	1630	1630	5.68	153	20	2.3	4.8	1.8	9	43	89	39.5
1	6465300	1.65	1.65	28.2	2.8	22.6	2.48	5.2	2.2	10	43	91	40
1	6465310	206	206	14.6	25.83	22.2	2.5	5.2	2.1	9.9	42	90	39.9
1	6465400	0.6	0.6	40.3	1.37	22.6	2.48	5.2	2.2	10	43	91	40
1	6465440	157	157	18.6	27.4	22.9	2.5	5.2	2.1	10	42	91	40
1	6465680	137	137	18.3	20.6	23	2.5	5.2	2.1	10	42	91	40
1	6677500	1570	1530	14.3	205.6	15	1.7	3.7	0.6	2.5	46	88	41.7
1	6685000	1020	1020	8.17	118.1	15.7	1.9	4.1	0.8	1.6	48	90	46.2
1	6762500	1361	1361	16	158	16	1.7	3.8	0.6	11	46	87	42
1	6763500	3307	3307	10.3	326	16.6	1.9	4	0.8	1.3	48	90	44.7
1	6767100	10.4	10.4	16.31	8.9	21.4	2.3	5.1	1.4	12	49	92	49.8
1	6767200	1.83	1.83	38.04	3.36	21.4	2.3	5.1	1.4	12	49	92	49.8

HYD REG	GAGE ID	A (mi^2)	Ac (mi^2)	S (ft/mi)	L (mi)	P (in)	I2.24 (in)	I50.24 (in)	SN10 (in)	T1 FAHR	T3 FAHR	T4 FAHR	EVAP (in)
1	[illegible]767300	19.3	19.3	13.71	12.17	21.4	2.3	5.1	1.4	12	49	92	49.9
1	[illegible]767400	40.4	40.4	9.29	21.5	21.3	2.3	5.1	1.4	12	49	92	49.5
1	6767410	80.4	80.4	7.97	27.53	21.4	2.3	5.1	1.4	12	49	92	49.7
1	6767500	229	229	5.17	87.04	21.7	2.45	5.2	1.45	12.4	49	92	49.9
1	6829700	9.06	9.06	24.5	5.52	17.9	2.1	4.9	1.2	13.6	51	94	54
1	6836500	360	350	8.6	75	19.4	2.3	5	1.2	14	51	94	54.9
1	6839200	6.74	6.74	35.1	3.75	21.2	2.3	5	1.3	12.2	50	92	51.1
1	6839400	13.2	13.2	28.7	7.9	21.3	2.3	5	1.3	12.2	50	92	51.1
1	6839600	11.3	11.3	31.2	5.1	21.3	2.3	5	1.3	12.1	50	92	51.9
1	6839850	13.8	13.8	36.4	5.2	20.8	2.4	5	1.3	11.6	49	92	49
1	6839900	31.8	31.8	22.1	11.6	20.9	2.4	5	1.3	11.7	49	92	49.1
1	6839950	25.6	25.6	20.5	15.2	20.8	2.4	5	1.3	11.5	49	92	49.1
1	6840000	74.3	74.3	15.6	19.9	20.9	2.4	5	1.3	11.8	49	92	49.2
1	6840500	21.6	21.6	25.1	11.9	21.2	2.4	5.1	1.4	11.8	49	92	49.5
1	6841500	52	52	11.7	28	21.4	2.41	5.1	1.4	12	50	92	51
1	6844000	246	246	6.9	53.8	21.8	2.42	5.2	1.4	13	50	93	51.6
1	68[illegible]5200	1510	1350	6.8	234.4	19.6	2.31	5	1.1	14.1	51	93.8	56.3
1	68[illegible]000	1950	1650	6.7	324	18.3	2.27	4.9	1.1	14.4	52	93.1	55.6
1	6847500	3740	3280	6.5	346	19.1	2.32	5	1.1	14.4	51	94	55.8
1	68[illegible]9600	22.9	22.9	11.2	10.24	24.9	2.5	5.4	1.4	14.6	51	93	54
1	6850000	129	125	8.49	55.5	22.5	2.5	5.4	1.4	14.8	50	93	51.9
1	6850200	15.6	15.6	15.64	13.5	22.7	2.6	5.6	1.4	15.1	50	93	52.4
1	6851000	177	56	5.59	68.4	23	2.5	5.4	1.4	14.4	50	93	50.7
1	6851100	64	18.4	3.58	27	23.3	2.6	5.5	1.4	14.6	50	93	51.1
1	6851200	105	27.6	3.58	32.6	23.3	2.6	5.5	1.4	14.6	50	93	51.1
1	6851300	11.5	8.2	11.41	8.46	23.1	2.6	5.5	1.4	14.7	50	93	51.1
1	6851400	128	47.6	3.81	45.9	23.3	2.6	5.5	1.4	14.6	50	93	51.1
1	6851500	279	190	5.28	75.9	23.3	2.6	5.5	1.4	14.8	50	93	51.3
1	6852000	39.2	39.2	9.8	27.2	24	2.7	5.7	1.5	14	50	92	51
1	6853000	0.75	0.75	47.71	1.73	24.1	2.7	5.7	1.5	14	50	92	50.1

HYD REG	GAGE ID	A (mi^2)	Ac (mi^2)	S (ft/mi)	L (mi)	P (in)	I2.24 (in)	I50,24 (in)	SN10 (in)	T1 FAHR	T3 FAHR	T4 FAHR	EVAP (in)
2	6454000	450	400	8.7	75.8	16	1.77	3.8	0.7	11	46	88	44.8
2	6454100	840	750	6.6	125	16	1.82	3.8	0.7	11	46	88	45
2	6454500	1400	1300	5.9	193	16.2	1.95	3.9	0.8	10	46	89	44.7
2	6457200	32.3	32.3	16	23.7	16.3	2.02	4.2	0.9	10	47	89	45
2	6457500	4290	3130	6.5	259	17.8	2	4.2	0.9	10	46	89	44.3
2	6459200	440	28	10.2	92.1	18.4	2.2	4.6	1.3	9	44	91	42.2
2	6459500	660	44	9.05	108.6	18.5	2.2	4.6	1.3	9	44	91	42
2	6460900	85	10	9.97	20.09	17.7	2.2	4.7	1.4	9	43	89	40
2	6461000	390	200	9.52	52.31	18	2.3	4.7	1.5	9	43	89	40
2	6461500	8090	3700	7.4	396	18	2.2	4.3	1.2	9	45	90	42.8
2	6462000	8390	3770	7.4	418	18.1	2.2	4.4	1.2	9	45	91	42.8
2	6462500	600	340	9.9	102	21	2.32	4.9	1.6	10	44	90	40.5
2	6463500	390	110	17.46	44.3	22.3	2.4	5	1.8	10	44	90	40.5
2	6465000	12100	6040	7.5	498	19.1	2.2	4.7	1.5	9	44	91	41.7
2	6465500	12600	6430	7.6	523	19.2	2.3	4.8	1.5	9	44	91	41.6
2	6478520	52.7	52.7	22.2	18	25	2.7	5.5	2.5	10	42	90	39
2	6678000	362	337	18.2	45.7	14.4	1.75	3.8	0.65	11.9	47	89	45.6
2	6679000	77.2	77.2	28.4	17.8	13.6	1.75	4	0.85	12	48	89	46
2	6687000	1190	80	7.9	132.1	16.6	2.1	4.4	1.1	11.7	47	90	46.3
2	6692000	940	80	10.5	81.7	18.5	2.24	4.8	1.3	10.2	47	90	45.5
2	6775500	1850	80	9.5	145	19.1	2.2	4.7	1.35	10	46	91	44
2	6775900	960	30	10.7	117	19.1	2.2	5	1.4	10.2	47	90	44.6
2	6776500	2040	45	11.7	147	19.1	2.2	4.8	1.3	10.2	47	90	44.9
2	6777000	3950	135	9.5	152	19.1	2.2	4.8	1.3	10.1	46	91	44.5
2	6777500	4650	430	9.9	179	19.5	2.2	4.8	1.3	10.3	46	90	44.4
2	6779000	5040	820	8.8	212	19.7	2.2	4.9	1.3	10.2	46	90	44.4
2	6780000	5310	1090	8.4	242	19.9	2.2	4.9	1.4	10.3	46	90	44.4
2	6782500	1570	890	5.8	189	21.3	2.4	5.2	1.6	11.6	47	91	45.6
2	6782700	86.1	45.9	8.6	21	22.1	2.4	5.2	1.6	11	46	90	45
2	6782800	15.5	10.8	33.6	11	22.3	2.41	5.2	1.6	11	46	90	44.9

HR	DG	GAGE ID	A (mi^2)	Ac (mi^2)	S (ft/mi)	L (mi)	P (in)	I2,24 (in)	I50,24 (in)	SN10 (in)	T1 FAHR	T3 FAHR	T4 FAHR	EVAP (in)
2		6784000	2350	1650	4.6	202	21.7	2.4	5.3	1.7	11.4	47	91	45.5
2		6785000	8090	3200	8.1	271	20.6	2.3	5	1.5	10.7	46	90	44.8
2		6786000	2280	180	7.6	185	20.6	2.3	4.8	1.5	9.4	45	91	42.1
2		6787500	1060	110	8.2	111	21	2.3	5.1	1.7	10.4	44	90	41.4
2		6788500	3750	770	7.1	221	21.1	2.3	4.9	1.6	9.7	45	90	42
2		6789000	3960	910	7	240	21.2	2.3	4.9	1.6	9.7	45	90	42.1
2		6790500	4290	1270	7	262	21.4	2.3	5	1.7	9.8	45	90	42.3
2		6791500	762	50	5.2	93	21.8	2.5	5.3	1.9	9.8	44	90	42.2
2		6792000	1220	480	4.42	148	22.6	2.5	5.3	1.9	9.9	44	91	42.5
2		6793000	14400	5650	7.5	318	21.2	2.3	5.1	1.6	10.4	46	91	43.8
2		6794000	647	410	4.5	126	24.6	2.7	5.4	2	9.9	44	92	42.2
2		6794500	15200	6230	7.1	342	21.4	2.4	5.1	1.6	10.4	46	91	43.7
2		6796978	128	64	11	34.2	21.5	2.4	5.1	1.9	10	43	90	41
2		6797500	1400	740	4.66	139.8	22.2	2.43	5.2	2	10.1	43	90	40.4
2		6798000	320	190	7.08	89.4	22.1	2.48	5.2	2	10.2	44	90	41.1
2		6798300	210	130	7.5	25.7	23	2.5	5.3	2	10	44	90	41.2
2		6798500	2200	1200	4.58	165.4	22.4	2.48	5.2	2	9	43	90	40.5
2		6799000	2790	1790	4.43	207.4	23	2.51	5.3	2	9.8	43	90	40.7
2		6800500	6900	5870	3.98	313.2	25.5	2.7	5.5	2.2	10.7	43	91	40.4
2		6821500	1640	980	17.4	148	16.7	2.1	4.4	0.9	13.2	52	90	51.5
2		6823000	1360	100	14.9	112	17.3	2.1	4.4	1	12.1	51	94	51.5
2		6823500	260	13	13.3	69	17.8	2.2	4.5	1.1	13.1	51	94	51.3
2		6824000	20	17	17.8	18.5	18	2.2	4.7	1.1	13.9	51	94	53.2
2		6831000	523	413	9.44	119	18.6	2.1	4.5	1.1	13.5	50	93	49.6
2		6831500	880	720	9.61	130	18.8	2.11	4.5	1.1	13.8	50	93	49.7
2		6835000	1500	380	10.7	119	19.9	2.23	4.7	1.2	13.1	49	93	49.3
2		6835100	30.2	30.2	27	9.36	19.5	2.2	4.8	1.25	14	51	93	54
2		6835500	2770	1470	8.99	205	19.7	2.18	4.7	1.2	13.5	50	93	49.8
2		6836000	320	270	12.4	54	21	2.3	49	1.3	13.8	49	92	51.8
2		6838000	830	410	11	91	20.5	2.3	4.9	1.3	13.2	50	92	50.6

HYD REG	GAGE ID	A (mi ^ 2)	Ac (mi ^ 2)	S (ft/mi)	I ([illegible]	P (in)	I2,24 (in)	I50,24 (in)	SN10 (in)	T1 FAHR	T3 FAHR	T4 FAHR	EVAP (in)
2	6839000	231	79	15	37.9	20.3	2.3	4.9	1.3	11.8	49	92	49
2	6839500	95	71	26.1	17.7	21.1	2.3	5	1.3	12.1	50	92	50.8
2	6841000	770	530	10.1	70.5	20.7	2.3	5	1.3	11.8	49	92	49.7
3	6465850	6.5	6.5	45.7	7.01	23.5	2.6	5.2	2.3	9.8	42	91	39.5
3	6466500	440	440	13.6	45.2	24	2.6	5.4	2.35	9	43	90	39.5
3	6478518	304	304	13.6	28.6	24	2.7	5.5	2.4	10	42	90	39.2
3	6600600	2.58	2.58	58.7	2.98	26.9	2.85	5.7	2.4	10	43	89	39
3	6600700	15.2	15.2	22.8	6.62	26.9	2.85	5.7	2.4	10	43	89	39
3	6600800	1.65	1.65	70	2.59	26.9	2.85	5.7	2.4	10	43	89	39
3	6600900	51.2	51.2	11.6	15.16	26.9	2.85	5.7	2.4	10	43	90	39.2
3	6601000	168	168	10.32	30.22	26.7	2.84	5.7	2.5	10	43	90	39.2
3	6607700	2.54	2.54	74.2	2.28	29.4	2.93	5.8	2.3	12	44	91	39.7
3	6607800	4.08	4.08	39.2	3.96	29.4	2.93	5.8	2.3	12	44	91	39.7
3	6607900	9.73	9.73	32	5.04	29.4	2.93	5.8	2.3	12	44	91	39.7
3	6608000	23	23	21.9	10.6	29.4	2.93	5.8	2.3	12	44	91	39.7
3	6608600	1.75	1.75	48.9	2.87	29	2.95	5.9	2.2	12	44	91	40
3	6608700	1.55	1.55	45.9	2.52	29	2.95	5.9	2.2	12	44	91	40
3	6608800	6.5	6.5	36.4	4.9	29	2.95	5.9	2.2	12	44	91	40
3	6608900	13.9	13.9	24.75	7.69	29	2.95	5.9	2.2	12	44	91	40
3	6609000	25.4	25.4	16.45	15.02	29	2.95	5.9	2.2	12	44	91	40
3	6610700	8.52	8.52	15.1	5.5	28	3	6	2.1	12	44	90.5	40
3	6795000	122	122	9.3	31.6	25.4	2.7	5.4	2	9.9	44	92	41.6
3	6795500	270	270	4.6	93	25.8	2.7	5.5	2	10.9	44	91	42
3	6799080	137	120	5.5	22.9	23.7	2.6	5.3	2.2	10	43	90	40.5
3	6799100	700	670	3.82	60	25.2	2.6	5.4	2.3	9.1	43	91	39.7
3	6799190	6.54	6.54	6.85	2.92	25	2.7	5.6	2.1	10	44	91	41.2
3	6799230	174	174	2.3	23.5	25	2.7	5.6	2.1	10	44	91	41.2
3	6799385	204	204	7.4	27.7	28	2.9	5.9	2.2	12	43	90	40
3	6799423	25.3	25.3	9	11.3	25.3	2.7	5.5	2.5	10	42	90	39
3	6799500	1030	1030	4.25	95.9	26.9	2.8	5.6	2.4	10.6	43	90	39.3

HYD REG	GAGE ID	A (mi^2)	Ac (mi^2)	S (ft/mi)	L (mi)	P (in)	I2,24 (in)	I50,24 (in)	SN10 (in)	T1 FAHR	T3 FAHR	T4 FAHR	EVAP (in)
3	6800000	450	450	5.23	75.5	27.7	2.8	5.7	2.1	12.3	44	90	41.2
3	6800350	6.53	6.53	4.5	6.7	28	2.9	5.9	2.2	12	44	91	40
3	6803000	167	167	5.69	32.99	28.2	3	6	1.7	14.9	48	92	44.5
3	6803500	684	684	3.93	58.99	28	3.01	6	1.7	14.4	48	92	44.1
3	6803510	43.6	43.6	3	13.7	27	2.8	5.8	2.3	12	43	90	40
3	6803520	47.8	47.8	10.6	11.3	27	2.8	5.8	2.3	12	43	90	40
3	6803530	119	119	11.6	18.6	27	3	6	1.9	12.2	46	92	42.5
3	6803540	7.88	7.88	22.19	5.01	28.7	3	6.1	1.8	14	47	92	42.9
3	6803555	1051	1051	3.5	77	28	3	5.9	1.8	14.3	47	92	43.7
3	6803570	0.43	0.43	85.71	1.2	29	2.91	5.8	1.8	13.3	46	92	42.7
3	6803600	15.4	15.4	26.98	6.83	29	2.9	5.8	1.8	13.1	46	92	42.5
3	6803700	9.09	9.09	26.44	6.35	29	2.9	5.8	1.9	13.2	46	92	42.6
3	6803900	43.3	43.3	11.95	16.6	29	2.9	5.8	1.9	13.2	46	92	42.5
3	6804000	271	268	5.98	38.53	29	2.93	5.9	1.9	13.3	46	92	42.1
3	6804100	7	6.7	10.24	4.31	29	2.9	5.9	1.8	13.1	45	91	41.4
3	6804200	30.3	23.4	7.87	10.65	29	2.93	5.9	1.9	13.1	45	91	41.5
3	6804300	10.3	7.3	8.33	7.59	29	2.9	5.9	1.9	13.1	45	92	41.5
3	6804400	17.6	14.2	7.66	13.31	29	2.92	5.9	1.9	13.1	45	92	41.6
3	6804500	80	63.4	6.74	24.68	28.8	2.95	5.9	1.9	13.3	45	92	41.6
3	6805000	1620	1590	3.33	87.8	28	3	5.9	1.8	14	46	92	43.1
3	6806400	20.8	20.8	11.17	10.06	29	3	6.1	1.8	12	47	92	42.5
3	6806420	5.23	5.23	21.13	3.33	29.2	3	6.1	1.8	12	47	92	42.6
3	6806440	10.3	10.3	15.42	5.95	29.2	3	6.1	1.8	12	47	92	42.6
3	6806460	80.1	80.1	8.03	29.44	29.2	3.1	6.2	1.8	12	47	92	42.4
3	6806470	0.73	0.73	97.56	1.64	29.7	3.1	6.2	1.8	12	47	92	42
3	6806500	241	241	6.48	52.03	29.8	3.1	6.2	1.85	12.5	47	92	42.2
3	6810060	3.43	3.43	13.3	3	32	3.2	6.5	1.7	16	47	92	41
3	6810100	8	8	16.77	8.84	28.8	3.03	6.1	43.2	14.5	47	93	43.2
3	6810200	59.6	59.6	9.81	23.03	28.9	3.04	6.1	1.8	14.5	47	92	43.2
3	6810300	25.5	25.5	14.73	13.7	30.1	3.07	6.2	1.8	14.5	47	92	42.7

HYD REG	GAGE ID	A (mi^2)	Ac (mi^2)	S (ft/mi)	L (mi)	P (in)	I2,24 (in)	I50,24 (in)	SN10 (in)	T1 FAHR	T3 FAHR	T4 FAHR	EVAP (in)
3	6810400	0.71	0.71	62.1	1.74	30.5	3.1	6.2	1.7	14.5	48	92	42.9
3	6810500	212	212	8.53	39	29.6	3.1	6.2	1.8	14.7	48	92	43.1
3	6811500	793	793	5.76	66.9	31	3.09	6.2	1.7	15	48	92	43
3	6814500	548	548	6.22	73.9	31	3.2	6.2	1.6	16	48	92	44.2
3	6815000	1340	1340	4.87	104.6	32	3.2	6.3	1.6	17	49	90	44.4
3	6815500	186	186	8.56	38.6	34	3.2	6.4	1.6	16.5	49	92	43
3	6815510	2.99	2.99	15.2	2.3	32	3.3	6.4	1.5	16	50	92	43
4	6768050	2.08	2.08	39.4	1.7	21.7	2.44	5.2	1.4	11.6	48	91	48
4	6768100	5.21	5.21	23.46	7	21.8	2.44	5.1	1.5	11.6	48	91	47
4	6768200	33.5	30.6	24.3	15.5	21.8	2.44	5.2	1.5	11.6	48	91	47
4	6768400	17.1	17.1	18.49	11.19	21.7	2.44	5.2	1.5	11.6	48	91	47
4	6768500	63	58.5	13.17	25.25	21.8	2.44	5.2	1.5	11.8	48	91	47.5
4	6769000	175	170	4.91	75.44	22.1	2.4	5.2	1.5	12	48	92	48.3
4	6769100	0.58	0.58	46.88	1.24	22.2	2.4	5.2	1.5	11.8	48	92	48
4	6769200	14.9	14.9	11.06	10.27	22.2	2.4	5.2	1.5	11.8	48	92	48
4	6770700	12.9	12.9	16.16	10.9	21.7	2.3	5.1	1.5	11.5	47	91	45.7
4	6770800	26.4	26.4	13.3	12.3	21.8	2.43	5.1	1.5	11.5	47	91	45.7
4	6770900	44.8	44.8	10.86	16.51	21.8	2.43	5.2	1.5	11.5	47	91	45.8
4	6770910	79.6	79.6	8.18	28.88	21.9	2.43	5.2	1.5	11.5	48	91	46
4	6771000	379	379	4.95	100.8	22.5	2.45	5.3	1.5	11.5	48	91	47.1
4	6771500	572	572	3.97	150.6	22.9	2.4	5.3	1.5	11.6	48	92	47.4
4	6772000	628	628	3.6	185.8	23	2.49	5.4	1.55	11.7	48	92	47.4
4	6777700	4.77	4.77	28	4.1	22.1	2.4	5.2	1.7	10.4	46	90	44.7
4	6778000	4790	475	9.1	190	19.6	2.2	4.9	1.3	10.3	46	90	44.4
4	6782600	0.4	0.4	40.5	1.56	22.2	2.41	5.2	1.6	11	46	90	44.9
4	6782900	5.9	5.9	56.6	4.94	22.6	2.41	5.2	1.6	11	46	90	45
4	6783500	707	655	5.9	105	22.5	2.45	5.3	1.7	11	47	91	45.2
4	6784300	41.9	41.9	9.9	20.1	23.2	2.5	5.3	1.7	10.1	46	90	44.5
4	6784700	27.2	27.2	12.9	15.7	23.4	2.6	5.4	1.8	10.7	46	92	44.9
4	6788988	58.5	58.5	6.35	13.7	20.5	2.5	5.4	1.7	10	46	90	43

HYD REG	GAGE ID	A (mi ^ 2)	Ac (mi ^ 2)	S (ft/mi)	L (mi)	P (in)	I2,24 (in)	I50,24 (in)	SN10 (in)	T1 FAHR	T3 FAHR	T4 FAHR	EVAP (in)
4	6789100	2.29	2.29	49.3	2	23.1	2.5	5.3	1.8	9.9	46	91	44.3
4	6789200	6.79	6.79	22.9	6	23.1	2.5	5.3	1.8	9.9	46	91	44.3
4	6789300	21.1	21.1	16.2	10.1	23.1	2.5	5.4	1.8	9.9	46	91	44.3
4	6789400	31.2	31.2	10	18.4	23.2	2.5	5.4	1.8	9.9	46	91	44.3
4	6790600	1.52	1.52	41.4	2.9	23	2.6	5.4	1.9	9.8	45	92	43.5
4	6790700	19.5	19.5	16.3	11.7	23	2.6	5.4	1.8	9.8	45	92	43.4
4	6790800	36.9	36.9	13.3	17.5	23.1	2.6	5.4	1.8	9.9	45	92	43.5
4	6790900	7.63	7.63	27.5	5.1	23.8	2.6	5.4	1.9	10.3	45	92	43.5
4	6791100	184	184	7.7	48.24	23.3	2.6	5.4	1.8	9.9	45	92	43.6
5	6794710	8.75	8.75	7.8	4.64	25.8	2.9	5.9	1.9	13	45	91	43
5	6880000	446	446	3	140.2	27	2.8	5.7	1.7	13.5	47	92	44.8
5	6880500	1101	1101	2.84	151.5	27	2.8	5.7	1.8	13.3	47	92	44.1
5	6880508	85.5	85.5	5.1	20.6	26	2.9	5.9	1.7	14	47	92	44
5	6880590	7.52	7.52	12.2	4.1	23	2.7	5.6	1.7	14	47	92.5	46
5	6880710	14.6	14.6	6.22	12.8	24.8	2.73	5.7	1.6	14	49	92	47.6
5	6880720	51.5	37.7	7.35	14.3	24.8	2.73	5.7	1.6	14.1	49	92	47.5
5	6880730	16.4	13.9	7.55	10.6	24.9	2.73	5.7	1.6	14.1	49	92	47.1
5	6880740	90.1	49.7	5.95	24.7	24.9	2.74	5.7	1.6	14.1	49	92	47.3
5	6880775	1.16	1.16	7.1	1.41	24	2.7	5.7	1.7	14	47	92	46
5	6880800	1206	1140	3.15	184.7	26.6	2.8	5.7	1.7	14.4	48	92	46.2
5	6881000	2716	2650	2.63	196.6	27	2.8	5.7	1.7	14	48	92	45.2
5	6881200	460	460	5.7	69.92	28.3	2.8	6	1.7	14.5	47	92	45.1
5	6881450	74.7	74.7	6.26	29.69	28.9	3.1	6.1	1.6	15.5	49	92	45.1
5	6881500	3900	3830	2.35	248.8	27.3	2.9	5.8	1.7	14.4	48	92	45.2
5	6882000	4444	4370	2.21	273.6	27.6	2.9	5.8	1.7	14.6	48	91	45.3
5	6883000	979	979	5.75	103.6	24.6	2.7	5.6	1.5	14.1	50	93	49.6
5	6883540	2.11	2.11	13.9	3.61	26	2.9	5.9	1.5	14.5	50	92	50
5	6883570	1552	1552	5	163.6	25.3	2.8	5.7	1.5	14.1	50	92	49.3
5	6883600	10.3	10.3	6.52	7.35	25.3	2.8	5.8	1.5	14	50	92	48.5
5	6883700	28.1	28.1	4.31	18.64	25.8	2.81	5.8	1.5	14	50	92	48.5

HYD REG	GAGE ID	A (mi ^ 2)	Ac (mi ^ 2)	S (ft/mi)	L (mi)	P (in)	I2,24 (in)	I50,24 (in)	SN10 (in)	T1 FAHR	T3 FAHR	T4 FAHR	EVAP (in)
5	6883800	50.4	50.4	4.27	28.99	25.9	2.81	5.8	1.5	14	50	92	48.5
5	6883900	90.3	90.3	4.01	53.18	26.3	2.9	5.9	1.5	14.2	50	92	48.5
5	6883955	11.6	11.6	10.6	4.72	26	2.8	5.9	1.6	15	47	92	48
5	6884000	2350	2350	4.86	184.8	25.6	2.9	5.8	1.5	14.4	50	92	48.9
5	6884005	4.51	4.51	23.4	3.21	27	3	6	1.5	15	50	92	50